IN-DELTA STORAGE PROGRAM
STATE FEASIBILITY STUDY

INTEGRATED FACILITIES ENGINEERING DESIGN AND ANALYSES

DRAFT REPORT

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Department of Water Resources
Division of Planning and Local Assistance
Surface Storage Branch
In-Delta Section

Division of Engineering
Mechanical and Electrical Engineering Branch

URS, Corporation

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Chapter 1: Introduction

1.1 General

In-Delta storage investigations were authorized under the CALFED Integrated Storage Investigations Program as defined in the CALFED Bay-Delta Program Record of Decision (ROD) and Implementation Memorandum of Understanding (MOU) signed by the CALFED Agencies on August 28, 2000. The ROD identified In-Delta storage as one of five surface storage projects to be studied. As a part of the In-Delta Storage investigations, the CALFED Agencies also decided to explore the lease or purchase of the Delta Wetlands (DW) Project, a private proposal by DW Properties Inc. to develop and market a water storage facility in the Sacramento-San Joaquin Delta (Delta). The proposed DW project included conversion of two islands, Webb Tract and Bacon Island, into "reservoir" islands and conversion of Bouldin Island and Holland Tract into "habitat" islands. The ROD included an option to initiate a new project if the DW Project proved cost prohibitive or technically infeasible.

The California Department of Water Resources and the CALFED Bay-Delta Program, with technical assistance from the U.S. Bureau of Reclamation, conducted a joint planning study to evaluate the DW project and other In-Delta storage options for contributing to CALFED water supply reliability and ecosystem restoration objectives. The main purpose of the investigations was to determine if the proposed DW project was technically and financially feasible. The joint planning study, completed in May 2002, concluded that the project concepts as proposed by DW were generally well planned. For ownership by DWR and USBR, however, the project as proposed by DW requires modifications and additional analyses before it is appropriate to "initiate negotiation with Delta Wetlands owners or other appropriate landowners for acquisition of necessary property" (CALFED ROD, page 44).

The re-engineered In-Delta Storage project has the same reservoir and habitat islands as the proposed DW project. The design modifications include a re-engineered embankment design around the reservoir islands and four consolidated inlet and outlet structures (integrated facilities); two on each of the reservoir islands. The project islands are shown in Figure 1.1.

This chapter includes a description of the re-engineered In-Delta Storage Project integrated facility concept, an outline of the scope of work completed, a summary of the integrated facility design features, a summary of the integrated facility structures cost estimates performed by CH2M HILL, and conclusions and recommendations. The hydraulic and structural engineering design and analyses conducted for the integrated facilities are discussed in detail throughout this report.

This report includes an analysis and cost estimate for a direct connection to Clifton Court Forebay. This direct connection cannot be justified due to costs outweighing the benefits, but it may be considered as a part of the newly proposed fish screens at CCF, reducing the required screen size of the proposed fish screens at CCF. With that said, the cost of this direct connection will not be added to the overall In-Delta Storage Project cost. Instead, the cost of this direct connection could be counted as an avoided cost of the proposed fish screens at the new CCF intake project, if deemed justifiable. Details on the design and cost of the direct connection to Clifton Court Forebay are provided in Appendix D.

1.1.1 Purpose and Need for In-Delta Storage

The purpose of In-Delta storage is to:

- help meet the ecosystem needs of the Delta,
- help achieve Environmental Water Account (EWA) and Central Valley Project Improvement Act (CVPIA) goals,
- provide water for use within the Delta, and
- increase reliability, operational flexibility and water availability for south of the Delta water use by the State Water Project (SWP) and the Central Valley Project (CVP).

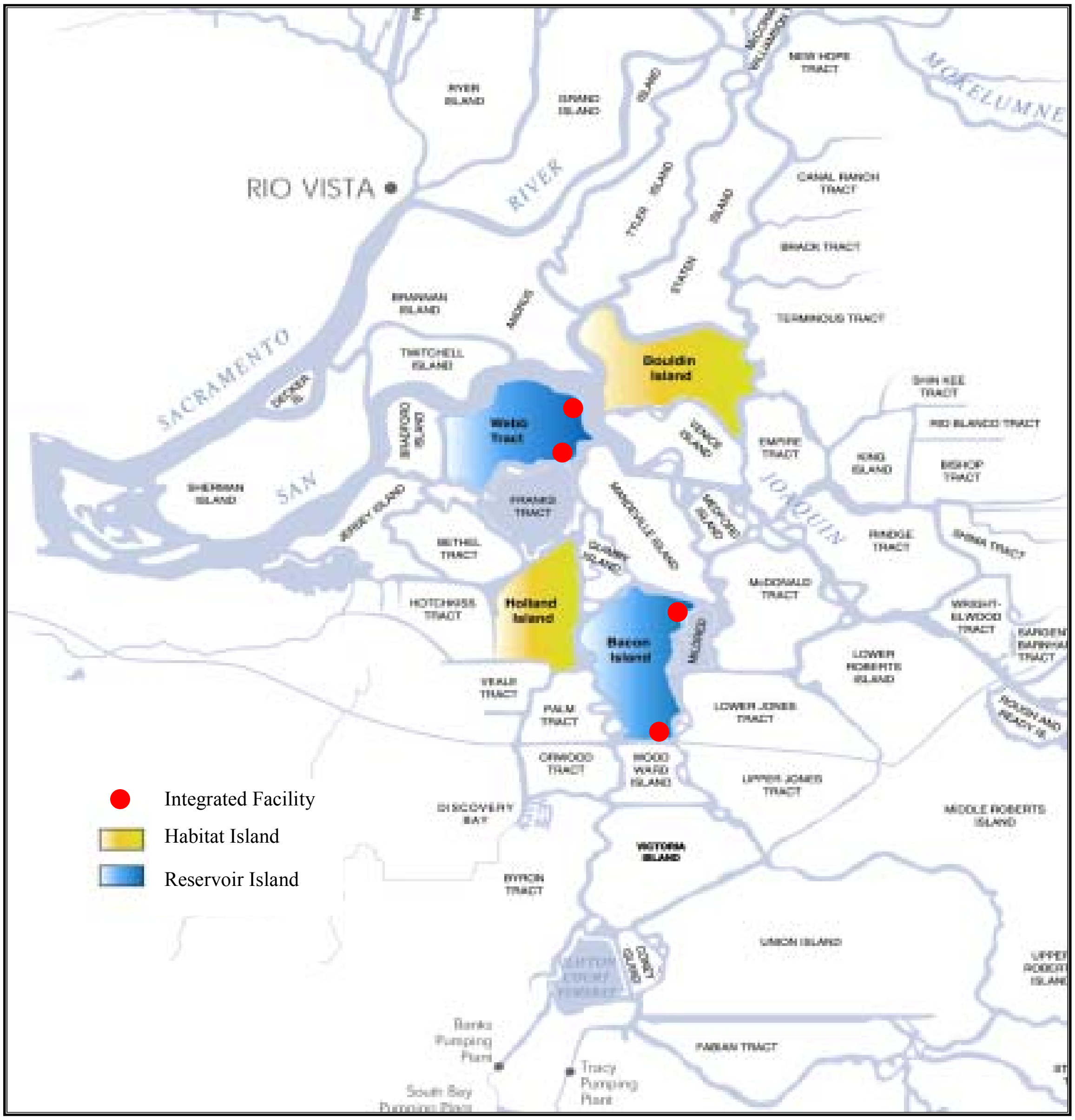


Figure 1.1 – In-Delta Storage Project Islands and Integrated Facility Locations

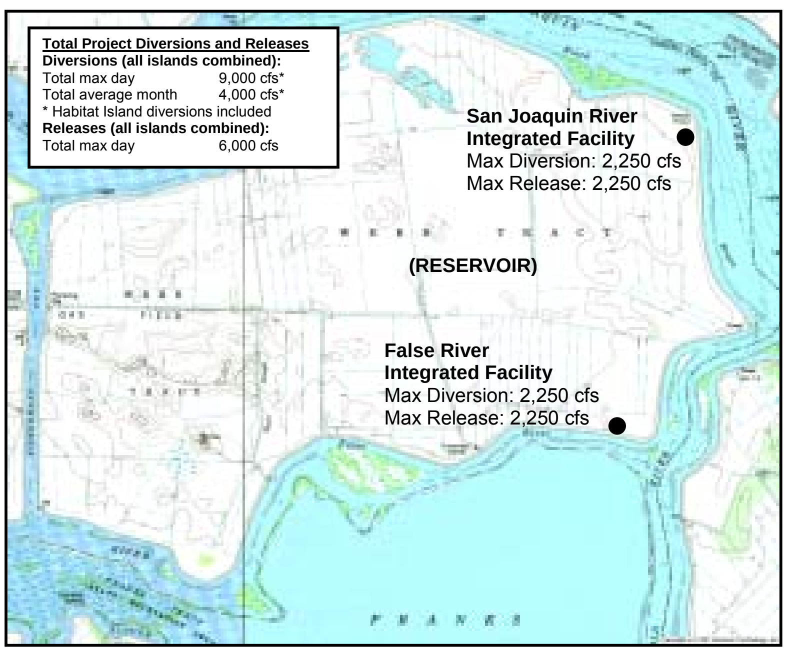
Improved operational flexibility would be achieved by providing an opportunity to change the timing of Delta exports and new points of diversion that could be selectively used to minimize impacts on fish. The In-Delta Storage Project would divert water from the Delta to Webb Tract and Bacon Island for storage during periods of high flow and low fish impacts. The stored water could be used to make up for export curtailments made during times most critical to listed fish species. New storage in the Delta could be useful to the California water system because it would:

- increase water supply reliability.
- improve system operational flexibility.
- allow reservoir space to be temporarily used for water transfers and banking.
- allow water to be stored and released to meet CVPIA and EWA goals and water quality constraints.
- allow surplus water to be stored during wet periods and when upstream reservoirs spill, permitting water to be stored in the Delta and released into the San Joaquin River and other in-stream channels for fisheries during dry periods.

1.2 Integrated Facility Concept

The DWR/USBR joint planning study made a number of recommendations. One recommendation was that solutions "be developed to enhance project reliability through improved design and consolidation of inlet and outlet structures." The consolidation of inlet and outlet structures will be achieved with integrated facilities.

Figure 1.2 – Webb Tract Integrated Facilities Location Map

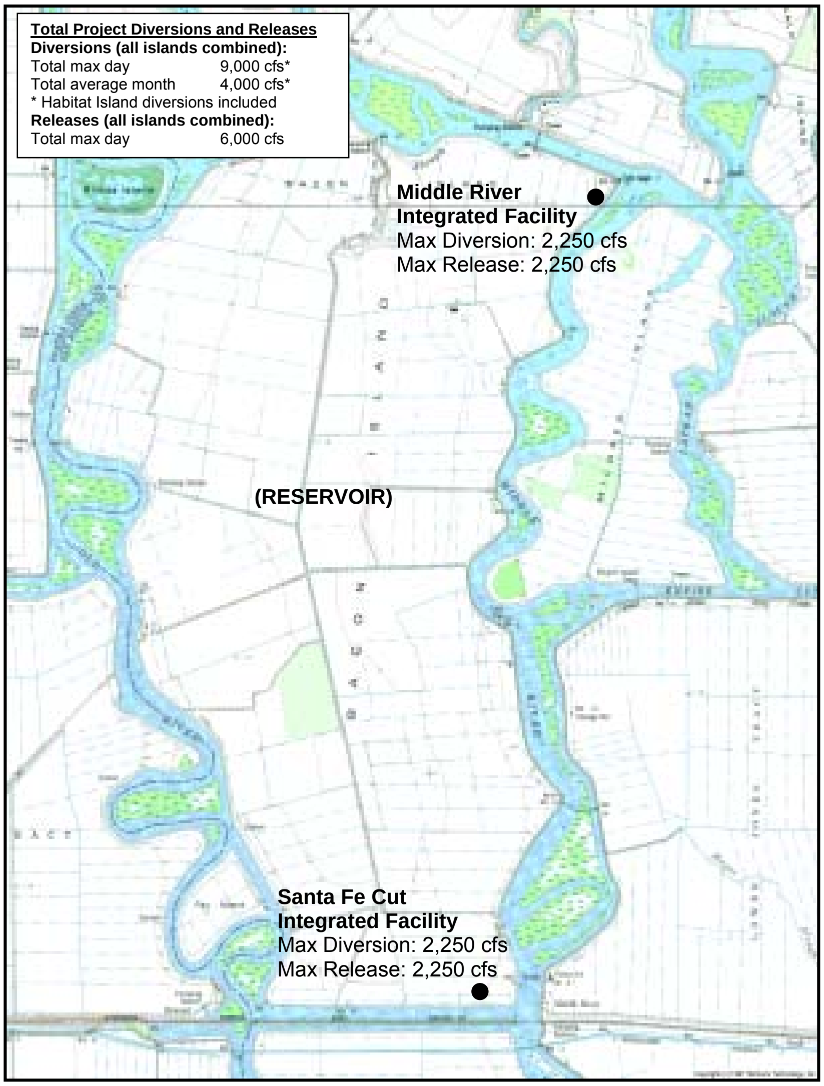


There are a total of four integrated facilities, two on Webb Tract and two on Bacon Island, and their locations are shown in Figures 1.2 and 1.3. The facilities will be used to control the diversion and release of water onto and off of the reservoir islands. The integrated facilities are consolidated control structures that combine all operational components into one facility. The operational components of each facility primarily include a fish screen, a transition pool, three inlet/outlet structures, a midbay, a pumping plant and associated conduits, a bypass channel and engineered embankments. Figure 1.4 shows the layout of a typical integrated facility and Figure 1.5 depicts a conceptual 3-dimensional illustration of a typical integrated facility. The overall goal of the integrated facility operations is to maximize gravity flow and minimize pumping to reduce operation and maintenance costs. All integrated facility components are described in more detail in sections 1.2.1 through 1.2.8.

The key features of each integrated facility are as follows:

- The fish screen is isolated from the other controls with a transition pool
- Storage diversions and releases can occur when the river and reservoir are at different levels, allowing for year-round operations
- Diversions and releases are optimized with gravity flow and pumping combinations
- Required flow under gravity is possible with small head differences
- Low midbay level and pumping units allow for complete drainage of reservoir when necessary

Figure 1.3 – Bacon Island Integrated Facilities Location Map



In order to complete a feasibility level design (state version) of the In-Delta Storage Project integrated facilities, hydraulic and structural design criteria and procedures were established and hydraulic analyses were performed to optimize sizes for the fish screen, inlet and outlet structures, pumping plant and conduits, and the bypass channel. Structural design was performed to determine sizes of the structural components. The design criteria and procedures established were typical for each integrated facility location, but separate hydraulic analyses and structural design were performed for each facility due to the varying conditions between facilities.

A high level of coordination to complete the feasibility level design of the integrated facilities was exercised among the In-Delta Section, DWR's Division of Engineering, USBR, various engineering consulting teams, and the Central Valley Fish Facilities Review Team (CVFFRT).

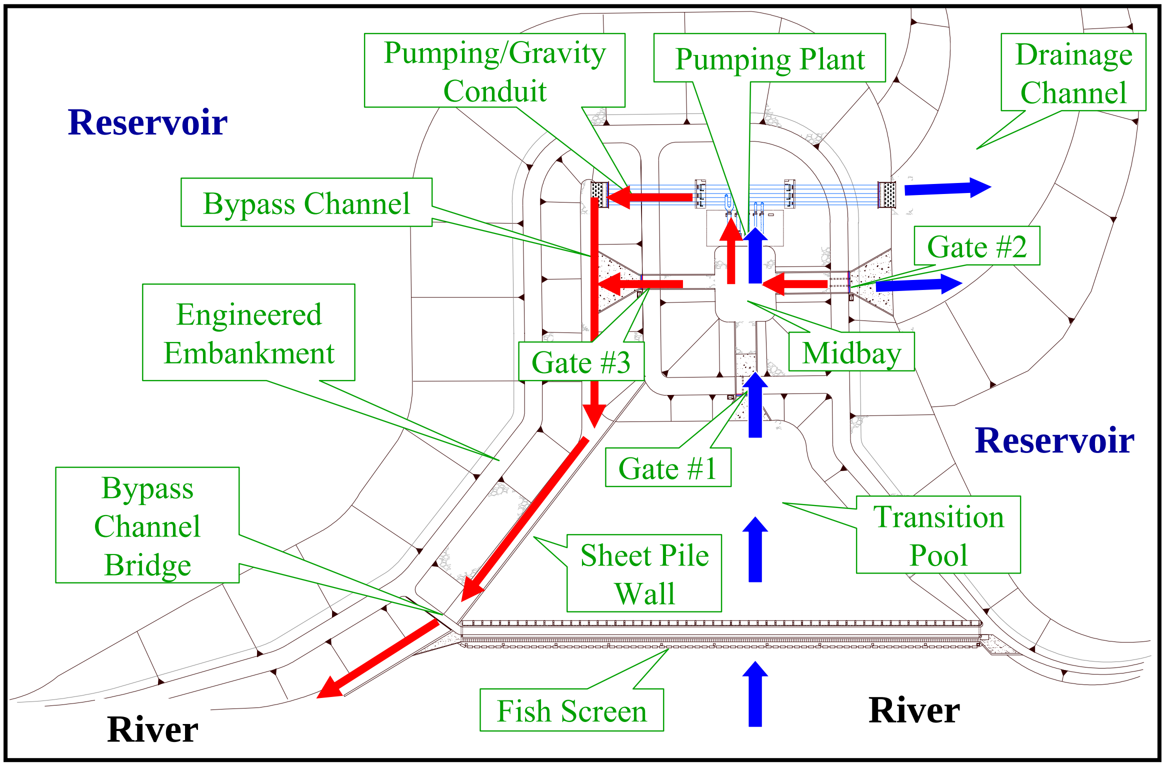


Figure 1.4 – Typical Integrated Facility Layout

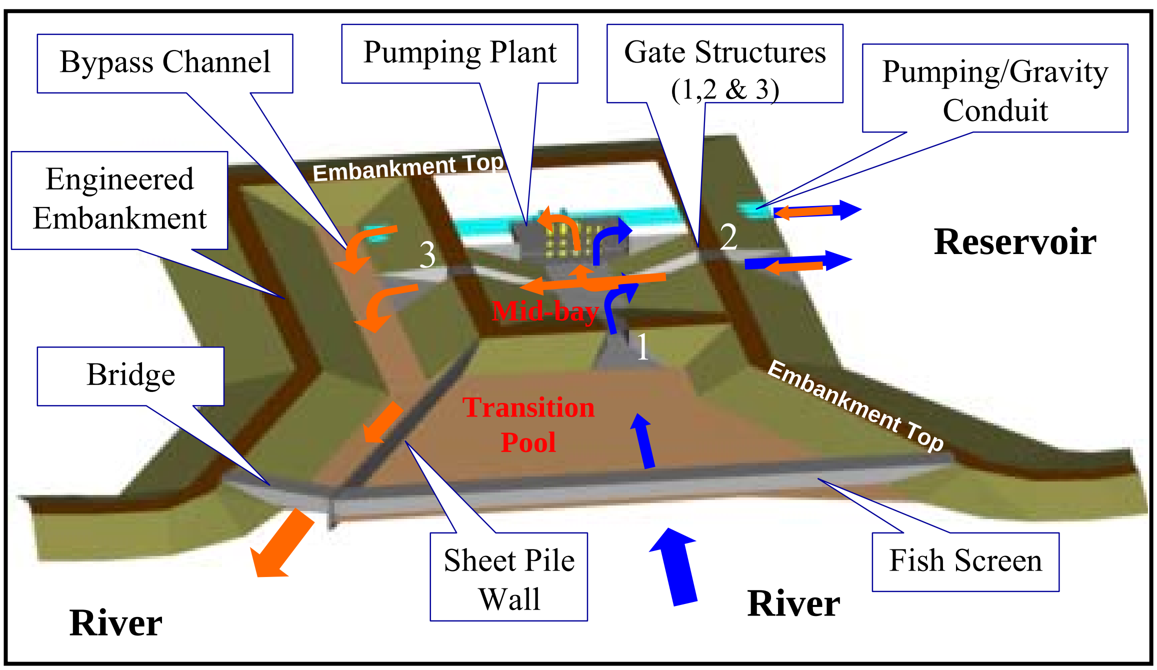


Figure 1.5 – Conceptual 3-Dimentional Illustration of a Typical Integrated Facility

1.2.1 Fish Screen Facility

As shown in Figure 1.4, the fish screen facility is located at the entrance to the integrated facility and is oriented adjacent and parallel to the river channel. The objective of the fish screen facility is to pass the design diversion rate over a range of water levels in both the river channel and the reservoir while protecting juvenile fish from entrainment, impingement and migration delay. The fish screen facility should operate with hydraulic efficiency and should not hamper the movement of fish species present in the river channel.

The proposed fish screens will be vertical profile bar type and will be continuously cleaned to prevent excessive debris buildup. The screens will meet applicable design criteria set forth by the California Department of Fish and Game (DFG) and the National Marine and Fisheries Service (NMFS). The total width of the fish screen facilities varies from 724 feet to 1,120 feet, depending on facility location. The fish screens will be placed above the river channel bottom to reduce the accumulation of sediments at the bottom of the screen. Further details of the proposed fish screen facility design can be found in Chapter 3.

CVFFRT has been briefed on the proposed fish screen design, facility layout, and overall operations. In response, CVFFRT gave a positive response to the proposed concept and suggested that a technical review team be assembled in the final design stage of this project.

1.2.2 Transition Pool

The transition pool, shown in Figure 1.4, is located immediately downstream of the fish screen facility. The purpose of the transition pool is to:

- separate the fish screen from the other operational controls,
- create a smooth transition of flow from the very wide section of the fish screen facility to the narrow section at Gate #1, and
- act as a settling basin to prevent excess suspended silt from entering the reservoir. The bottom of the transition pool will not be lined so that it can be dredged periodically to remove accumulated sediments.

1.2.3 Gate Structures

Each integrated facility consists of three gate structures, which are shown in Figure 1.4. Each gate structure, as described below, serves a unique purpose in the integrated facility operations.

Gate #1 is used strictly during diversion operations to regulate flows into the midbay. Once the water enters the midbay, it is routed into the reservoir either by gravity flow through Gate #2, by pumped flow through the pumping plant or by a combination of both gravity and pumped flow. Fluctuating water levels in both the river and the reservoir will dictate the need for and extent of pumping during diversion operations.

As mentioned previously, Gate #2 is used to regulate the flow of water from the midbay to the reservoir during diversion operations. Gate #2 can also be used to regulate the flow of water out of the reservoir and into the midbay during release operations.

Gate #3 is used strictly during release operations to regulate flows from the midbay into the bypass channel. Once the water enters the midbay from Gate #2 it is routed into the bypass channel either by gravity through Gate #3 by pumped flow through the pumping plant or by a combination of both gravity and pumped flow. Fluctuating water levels in both the river and the reservoir will dictate the need for and extent of pumping during release operations.

1.2.4 Midbay

The midbay is located at the center of the integrated facility gate structures and pumping plant as shown in Figure 1.4. The midbay serves as a flow regulation pool during diversion and release operations. It also serves as a forebay for the pumping plant when it is operating. The midbay floor is at a low enough elevation to allow complete drainage of the reservoir while providing adequate submergence of the pumping units. The size of the midbay will allow for a smooth transition of flow during all operations and will be sufficient to facilitate maintenance as required.

1.2.5 Pumping Plant and Conduit

The pumping plant is located adjacent to the midbay on the side opposite to Gate #1 and the conduit pipes stretch from the reservoir side of the integrated facility to the bypass channel, as shown in Figure 1.4. Determination of pump sizes, and hydraulic calculations are described in Chapter 5.

The pumping plant serves two main purposes: (1) to supplement diversion and release gravity flows when sufficient head is not available to meet the desired flow rates and (2) to meet the desired flow rate when the net head is zero or negative. The pumping plant consists of five pumping units, three pumps with a capacity of 400 cubic feet per second (cfs) each and two pumps with a capacity of 150 cfs each, totaling a maximum pumping capacity of 1500 cfs. This combination of pump sizes will allow flexibility in operations when needed. The smaller pumps have lower submergence requirements than the larger pumps and can be used to pump water out of the reservoir at lower elevations. All pumping units are assumed to be vertical, mixed flow pumps driven by a motor connected to a right angle gear. A formed suction intake (FSI) will be mounted to each pump below the impeller to eliminate vortex formation in front of the pump.

During diversion operations the pumps will be used to supplement gravity flow into the reservoir through Gate #2 when there is insufficient head available to achieve the target flow rate via gravity through Gate #2. When the water level in the reservoir is equal to or exceeds the water level in the river, Gate #2 will be closed and the desired flow rate will be met by pumping only. The pumped flows will be routed through the conduit pipes and discharged into the reservoir.

During release operations the pumps will be used to supplement gravity flow from the reservoir through Gate #3 when there is insufficient head available to achieve 100 percent of the desired flow rate via gravity through Gate #3. When the water level in the river (or bypass channel) is equal to or exceeds the water level in the reservoir (or midbay), Gate #3 will be closed and the desired flow rate will be met by pumping only. Under this scenario, Gate #2 will remain open to allow water to enter the midbay. The pumped flows will be routed through the conduit pipes and discharged into the bypass channel.

The conduit pipes are used to discharge water into the reservoir and bypass channel during diversion and release pumping operations, respectively. For both operational scenarios the flow direction is controlled by two butterfly valves installed in each conduit pipe. For diversions to be made through the pumping plant, the valve closest to the bypass channel is closed and the valve closest to the reservoir is opened; the opposite is true for releases. The two baffled apron structures, located at each end of the conduit pipes, serve as energy dissipaters during both diversion and release operations when pumping is required as well as when the conduit pipes are used for gravity flow releases. The conduit pipes can also be used for gravity flow releases to supplement the gravity flow releases through Gate #3. This can be achieved by opening both butterfly valves in each conduit pipe.

Stop log slots will be provided in front of each pumping plant intake. This will allow individual pumping units to be shut down and serviced while the rest of the units continue operating. A gantry crane will also be provided to facilitate required maintenance and inspections.

1.2.6 Bypass Channel

The bypass channel is used to convey reservoir releases into the river and is shown in Figure 1.4. Reservoir releases enter the bypass channel at its upstream end through the conduit pipes and/or through Gate #3. The bypass channel is isolated from the fish screen facility and transition pool by a structural sheet pile wall. The embankments surrounding the bypass channel will be lined with rock riprap as required to prevent bank erosion.

There will be a vehicle access bridge spanning the bypass channel connected on one end to the integrated facility embankment and on the other end to the fish screen structure. This bridge will allow access to the fish screen from both ends as well as allow traffic to move from one side of the facility to the other. The bridge will support a trash rack on the downstream end to prevent debris in the river from entering the bypass channel. To prevent the attraction and egress of adult-sized fish (specifically salmon and steelhead) from the Delta channels, the trash rack will be designed to prevent fish passage into the bypass channel and midbay area where they could be trapped.

The interior reservoir will not be managed for a fishery but, since eggs and larvae will likely enter the island through the fish screens, a fishery may be present. Predator-sized fish should be kept from returning to the adjacent channels if possible. A trash rack will be installed on the reservoir side of the facility in front of Gate #2 and in front of the conduit to prevent larger fish escapement from the island. The hydraulic design methodologies of the bypass channel are described in Chapter 6.

1.2.7 Embankments

Engineered embankments will surround the integrated facility on the reservoir side and will surround the midbay on all sides. The embankments are shown on Figure 1.4. All interior integrated facility embankments will have 3H:1V side slopes. All reservoir side (exterior) integrated facility embankments will also have 3H:1V slopes that will transition into the island embankments, which have 3H:1V side slopes from the embankment crest to elevation +4 feet above mean sea level (MSL) and a 10H:1V slope from +4 feet MSL downward. The embankment crest elevation varies for each integrated facility location, ranging from 10.2 and 10.4 feet above MSL at the Bacon Island facilities to 11 feet above MSL at the Webb Tract facilities. Riprap slope protection will be placed on all integrated facility embankments to avoid erosion from wind-wave action that could lead to embankment failure.

1.2.8 Operations and Maintenance

Table 1.1 shows the various diversion and release control combinations for integrated facility operations.

Table 1.1: Integrated Facility Diversion and Release Controls

Condition	Method	Gate Location			Intake/Discharge Conduit Valve Locations		
		Gate #1 [1,2]	Gate #2	Gate #3	Reservoir Side	Pumping Plant Discharge	Bypass Channel Side
Diversion	Gravity Only	Open	Open	Closed	Closed	Closed	Closed
Diversion	Combination	Open	Open	Closed	Open	Open	Closed
Diversion	Pumping Only	Open	Closed	Closed	Open	Open	Closed
Release	Gravity Only	Closed	Open	Open	Open	Closed	Open
Release	Combination	Closed	Open	Open	Closed/ Open [3]	Open/Closed [3]	Open
Release	Pumping Only	Closed	Open	Closed	Closed	Open	Open

1 – Gate #1 Open - Pool level is approximately the same level as the transition pool level upstream of Gate #1
2 – Gate #1 Closed - Pool level is approximately the same level as the reservoir level
3 – When releasing water using a combination of gravity flow and pumped flow, one or more of the conduit pipes may be used to release flows by gravity (from the reservoir to the river) provided that these conduit pipes are not used for pumping. In other words, any given conduit can either be used for gravity flow or pumped flow, but not both at the same time.

Periodic maintenance may be required for various structural components of the integrated facility. To perform maintenance of structural components that are normally under water, dewatering will be required. Stop log slots, along with stop logs, will be provided in front of each gate structure and at each end of the conduit pipes. With gate structure stop logs in place and the midbay dewatered, maintenance can be performed on the gates. Similarly, with stop logs in place at the conduit pipe outlets, maintenance can be performed on the conduit pipes. To perform maintenance on the fish screen structure, one bay can be dewatered at a time by placing stop logs in the rear stop log slots and by dropping the steel face wall at the front of the screen downward. In addition, to perform maintenance in the transition pool area (such as dredging), stop logs can be placed in all rear stop log slots of the fish screen structure and the transition pool can be dewatered.

1.3 Scope of Work

As a component of the overall In-Delta Storage Program engineering investigations, the purpose of this investigation was to provide feasibility-level designs of the four integrated facilities. The scope of work related to the integrated facility design included performing field investigations, design investigations, construction methods analysis, and cost estimation for the integrated facility components. This section outlines the work that has been completed.

1.3.1 Field Investigations

Field investigations that were conducted as part of the In-Delta Storage Program State Feasibility Study and used in the integrated facility design are as follows:

- Under Phase I of the In-Delta Storage Geologic Exploration Program, USBR conducted Cone Penetrometer Test (CPT) borings within the interior of Webb Tract and Bacon Island during August and September 2002.
- Under Phase II of the In-Delta Storage Geologic Exploration Program, DWR drilled four boreholes, one for each integrated facility location, to obtain samples for laboratory testing.
- All flow, water level and tidal data used in this design investigation was obtained from the Interagency Ecological Program data records.

1.3.2 Design Investigations

DWR developed design criteria, conducted hydraulic analyses, and prepared designs for the integrated facility fish screens, inlet and outlet structures, pumping plants and conduit pipes, and conveyance channels. CH2M HILL, with its subcontractor, URS Corporation, prepared State feasibility level structural analysis and design of the integrated facility fish screens, inlet and outlet structures, pumping plants and conduit pipes, sheet pile walls, conveyance channels, and bridge structures. The structural design and analysis is documented in Appendix B of this report.

1.3.3 Construction Methods Analysis and Cost Estimation

URS and CH2M HILL conducted an analysis on construction methods to be used in the construction of the integrated facilities. A representation of the optimum construction task sequence was also generated. This work is documented in the report titled "In-Delta Storage Program Integrated Facility Construction Cost Estimate," by CH2M HILL (2003).

URS and CH2M HILL estimated the quantity of materials needed to construct the integrated facility fish screens, inlet and outlet structures, pumping plants and conduit pipes, sheet pile walls, conveyance channels, and bridge structures. They then prepared a feasibility level cost estimate for the four integrated facilities. Supplemental information on equipment and costs was provided to URS and CH2M HILL by DWR. This work is documented in the report titled "In-Delta Storage Program Integrated Facility Construction Cost Estimate," by CH2M HILL (2003).

1.4 Summary of Integrated Facility Design Features

Table 1.2 provides a summary of the integrated facility design features.

Table 1.2: Summary of Integrated Facility Design Features

Integrated Facility Component	Bacon Island		Webb Tract	
	Middle River	Santa Fe Cut	San Joaquin R.	False River
Fish Screens				
Number of Bays	40	51	40	33
Length of Screens (ft)	800	1020	800	660
Total Width (ft)	878	1120	878	724
Gated Structures				
Design Inflow (cfs)	2250	2250	2250	2250
Design Outlow (cfs)	2250	2250	2250	2250
Inlet (Slough to Midbay)	3 Gates-12x10	3 Gates-12x10	3 Gates-12x10	3 Gates-12x10
Inlet/Outlet (Reservoir/Midbay)	3 Gates-12x10	3 Gates-12x10	3 Gates-12x10	3 Gates-12x10
Outlet (Midbay to Bypass)	2 Gates-12x8	2 Gates-12x8	2 Gates-12x8	2 Gates-12x8
Pumping Plants				
No. of Pumps and Size	2 – 150 cfs	2 – 150 cfs	2 – 150 cfs	2 – 150 cfs
	3 – 400 cfs	3 – 400 cfs	3 – 400 cfs	3 - 400 cfs
Plant Capacity (Total cfs)	1500 cfs	1500 cfs	1500 cfs	1500 cfs
Conduit Sizes (No. & Diameter)	2 – 8 ft	2 – 8 ft	2 – 8 ft	2 – 8 ft
Bypass Channel				
Bottom Width (ft)	40	70	30	30
Side Slopes	3:1	3:1	3:1	3:1
Embankments				
Crest Width (ft)	35		35	
Top of Embankment (El. ft)	10.2		11	
Slopes – Slough Side	3:1		3:1	
– Reservoir Side	3:1		3:1	

1.5 Summary of Integrated Facility Structures Cost Estimates

Feasibility level cost estimates for the In-Delta Storage Project integrated facility structures have been completed by CH2M HILL as a component of the overall In-Delta Storage Program engineering investigations. The cost estimates for each facility are summarized in Table 1.3. These costs do not include any contingencies and are broken down further in the *In-Delta Storage Program Integrated Facility Structures Construction Cost Estimate* report by CH2M HILL.

Table 1.3: Integrated Facility Structures Cost Summary

Item	Webb Tract		Bacon Island	
	San Joaquin R.	False River	Middle River	Santa Fe Cut
1. General Requirements	$4,232,477	$4,232,477	$4,232,477	$4,232,477
2. Sitework	$6,566,432	$6,532,868	$6,561,410	$6,628,637
3. Bypass Channel Bridge	$552,842	$484,309	$512,746	$521,782
4. Fish Screen	$9,014,880	$7,828,434	$8,967,133	$10,881,645
5. Gate #1 w/Wingwalls	$1,510,411	$1,441,773	$1,512,536	$1,416,896
6. Gate #2 w/Wingwalls	$1,801,695	$1,600,588	$1,770,101	$1,725,923
7. Gate #3 w/Wingwalls	$1,333,762	$1,212,493	$1,267,685	$1,187,624

8. Gate Control w/HPU Vault	$243,097	$237,784	$245,470	$243,770
9. Pumping Plant	$7,176,304	$7,142,776	$7,191,915	$7,180,763
10. Water Conduits	$1,407,796	$1,352,539	$1,433,583	$1,415,164
11. Valve Vaults	$2,726,803	$2,683,460	$2,738,267	$2,724,565
12. Baffled Apron	$264,199	$252,766	$261,183	$256,610
Subtotal for each Facility	**$36,830,697**	**$35,002,266**	**$36,694,504**	**$38,415,855**
Subtotal (without contingency)	**$146,943,322**			
Say	**$147,000,000**			

1.6 Conclusions and Recommendations

After completing feasibility level designs of the integrated facilities, the following conclusions and recommendations have been developed:

- CVFFRT has evaluated the proposed fish screen facilities and agrees with the overall concept. DWR should organize a technical review committee for fish screen review during the final design phase.
- Sensitivity studies should be conducted to optimize the configuration, size, and elevation of the inlet and outlet structures, the pumping plant, and the conduit pipes.
- The design and layout of the integrated facilities is considered to include sufficient detail for a feasibility level assessment of cost. Physical hydraulic model design studies should be conducted during final design of the integrated facilities. This, along with a technical fish screens review committee, will be important to finalize the design of all integrated facility components and to determine the specific setback location from the existing levee alignment.
- Given the planned configuration of the pumping plant, a partial vacuum may form within the piping downstream of the pumps when the pumps are shut down. An analysis should be performed to determine the maximum vacuum that may occur and the pipe thickness should be sufficient to avoid collapse. This analysis will require dynamic modeling.
- An area assessment should be performed by PG&E to develop accurate distances to the nearest utility source that can handle the In-Delta Storage project’s anticipated load and to determine the feasibility and cost associated with connecting power to the integrated facility sites.
- Additional analysis should be performed to refine the design of the conduit pipe outlet structures.
- Further structural engineering studies should be conducted to refine the design and extent of piles needed to support the integrated facility structures. The amount and extent of piles required may be reduced since the peat soils will be removed in the vicinity of the integrated facilities.

Chapter 2: Engineering Design and Analysis

2.1 Introduction

For a feasibility level design of the integrated facility components, geotechnical investigations and hydrology, hydraulic and structural design studies were conducted. Geotechnical investigations were conducted to determine the foundation conditions at each integrated facility location. Flow conditions were analyzed through a statistical analysis of recorded flow data to determine the stage variations in the channels adjacent to the integrated facilities. Numerous hydraulic analyses were conducted and structural design was performed to determine the overall layout and sizes of the integrated facility components. The structural design report was competed by CH2M HILL and URS Corporation (April 2003) and is provided in Appendix B.

This study used aerial photographs and survey data of existing levee crown elevations developed in 2001. Five-foot contour maps for Webb Tract, Bacon Island and Victoria Island were then developed from these surveys and were used in the design of the integrated facilities.

This chapter provides information on the overall study approach; while detailed analyses for each integrated facility component is presented in the chapters that follow.

2.2 Geological Investigations

Geologic explorations for potential borrow sources and integrated facility foundation evaluations were conducted on Webb Tract and Bacon Island by DWR's Division of Engineering, Project Geology Section and USBR. The geologic explorations were in two phases. Phase I consisted of CPT borings ranging from about 30 to 101 feet in depth. Shallow CPT soundings of 28 to 52 feet in depth were used for the characterization of borrow areas and materials on both islands, while deeper, 85 to 101 feet deep, soundings were used in the determination of foundation conditions beneath the proposed integrated facilities. Phase II of the investigation consisted of drilling and sampling one 100 foot drill hole at each of the four integrated facility sites. Locations of CPTs and drill holes are shown in the report titled "In-Delta Storage Program Borrow Area Geotechnical Report", by URS (April 2003).

Phase I Exploration

Under Phase I of the In-Delta Storage Geologic Exploration Program, USBR conducted CPT borings within the interior of Webb Tract and Bacon Island during August and September 2002. Ten CPT soundings, ranging from 85 to 101 feet deep, were used to determine foundation conditions at the four integrated facility locations. Six CPTs, CPT No.s WSC-11, -13, -15, -16, -17, and -18, were performed on Webb Tract (three at each integrated facility location) and four CPTs, CPT No.s BSC-1, -2, -12, and -13, were performed on Bacon Island (two at each integrated facility location).

The CPT data was recorded in the field and sent to the USBR office in Denver for processing. Processing the CPT tip resistance and sleeve friction data using the CPTINTR1 program, USBR determined a geologic log of Robertson's (1990) soil behavior types and basic engineering parameters, including undrained strength, The Standard Penetration Test (SPT) blow count equivalent, friction angle, and relative density for soils at each sounding location.

Phase II Exploration

Under Phase II of the In-Delta Storage Geologic Exploration Program, DWR drilled four boreholes (100 feet deep), one for each integrated facility location, to obtain samples for laboratory testing and to determine foundation conditions at the four integrated facility locations. The Webb Tract boreholes are labeled WTS-1 and WTS-2 and the Bacon Island boreholes are labeled BIS-1 and BIS-2.

Each boring was geologically logged and sampled by DWR. Shelby tube soil samples were collected from each boring at five-foot intervals below 30 feet in depth, and at 10-foot intervals from 30 to 100 feet. Immediately following the Shelby tube sample collection, a 1.38-inch inside diameter Standard Penetration Test sample barrel was driven 18 inches using a CME automatic hammer (140 lb.) set at 50-55 blows per minute with a 30-inch drop pursuant to ASTM D-1586 and D-6066. Uncorrected SPT "N" values are reported on the geologic logs for each samples interval.

Post Exploration Analysis

After completion of the Phase I and Phase II field work, the CPT and bore-hole logs were compiled and used to develop geologic cross sections and isopach maps showing the thickness of soft and/or organic soils overlying potential borrow materials. More information on these geologic explorations can be found in the "In Delta Storage Program; Results of Geologic Exploration Program" memorandum report (Project Geology Report No. 94-00-20) dated January 8, 2003.

Laboratory testing was then conducted on samples from the integrated facility locations by the DWR Bryte Soils Laboratory with guidance from DWR's Division of Engineering, Dams and Canals Section. SPTs and bag samples obtained during drilling provided disturbed samples, and Shelby tube sampling provided undisturbed samples. All samples were sent to the DWR Bryte Soils Laboratory for soil classification (particle size distribution and Atterberg limits), organic content determination, consolidation, and consolidated undrained triaxial tests with pore pressure measurements. Based on the geology logs, soils were classified into three primary groups: clay, silty sand, and organic soils. Detailed information on this laboratory testing can be found in the "In Delta Storage Program; Results of Laboratory Testing Program" memorandum report dated January 28, 2003.

2.3 Hydrology Investigations

The integrated facilities are located in tidally influenced areas so, in addition to seasonal and annual variations, river stage varies hourly. During tidal cycles, river flows vary both in quantity and direction. The diversion capacity of each integrated facility is 1500 cfs for low stage operations and 2250 cfs for high stage operations, and the release capacity is 1500 cfs. The facilities will operate year-round and should be able to deliver the design discharge under low flow as well as high flow conditions. Therefore, to perform hydraulic analyses and design of the integrated facilities, information about river stage variation is required. As described in this section, detailed statistical analyses of the available stage data were conducted to obtain historical distributions of the tidal stages near the integrated facility locations.

Tidal Analyses of River Stages

A DSM2 computer model simulation study was carried out to determine the stage variations in the channels adjacent to the intake facilities. The simulation period covered the period of water year 1974-1991 using historical hydrology and tidal boundaries. The output of the DSM2 model is in terms of hourly stage variations at each intake site. The DSM2 node numbers representing the intake sites are given in Table 2.1.

Table 2.1: Intake Facility and DSM2 Channel Output Locations

Intake Facility	DSM2 Channel Node No.
Webb Tract at San Joaquin River	44
Webb Tract at False River	278
Bacon Island at Middle River	153
Bacon Island at Santa Fe Cut	144

For each day, high-high, low-high, low-low and high-low stages (see Figure 2.1 for definition) were extracted. For each series, statistical analyses were carried out to determine the mean, median, minimum, maximum and standard deviation of stages. The results of these analyses are summarized in Tables 2.2(a) through 2.2(d).

Frequency analyses of the time series were carried out to determine the stages having different levels of probability of occurrences. The probability plots of the stages are shown in Appendix A, Figures A.1 through A.4. Tidal stages and the probability occurrences are summarized in Table 2.3.

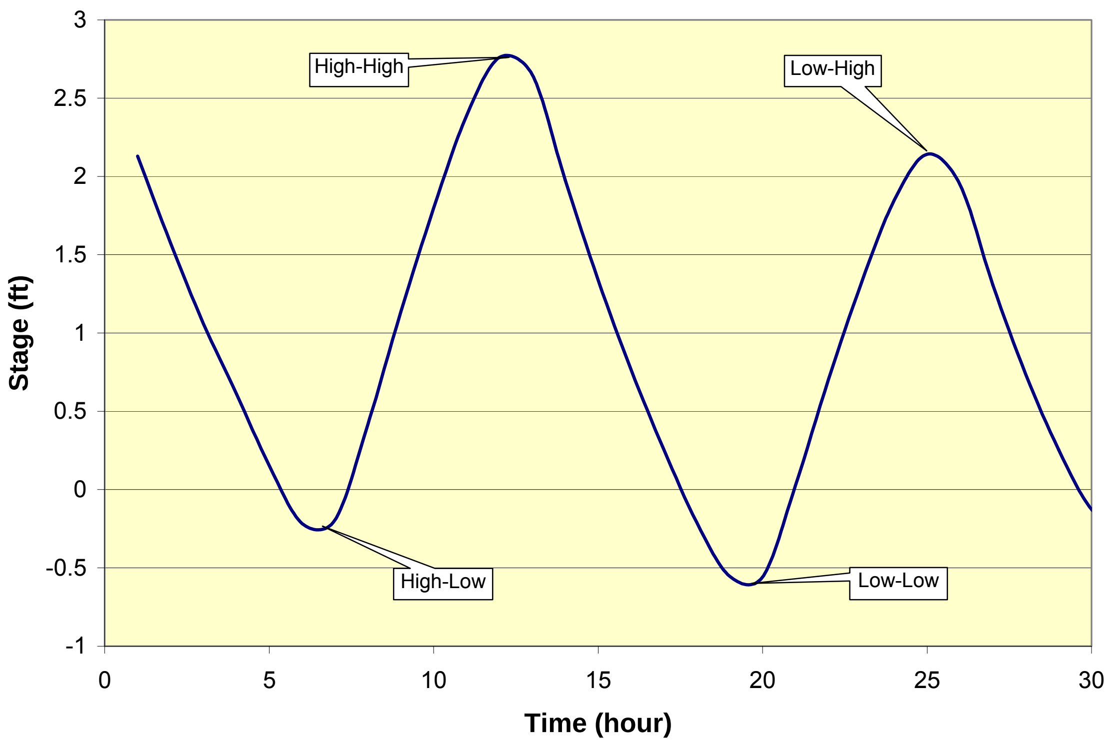


Figure 2.1: Definition Figure for Tidal Stage

Table 2.2: Summary of Statistical Analyses of Stage Time Series

(a) Webb Tract at San Joaquin River

	Stage (ft)			
	HighHigh	LowHigh	LowLow	HighLow
Maximum	6.826	5.810	5.003	5.003
Minimum	1.331	0.639	-1.714	-1.398
Mean	3.109	2.237	-0.651	0.125
Median	3.080	2.129	-0.768	0.004
Std. Dev	0.615	0.527	0.552	0.641

(b) Webb Tract at False River

	Stage (ft)			
	HighHigh	LowHigh	LowLow	HighLow
Maximum	6.36	5.57	4.21	4.87
Minimum	0.91	0.50	-1.54	-1.30
Mean	2.95	2.05	-0.44	0.36
Median	2.93	1.98	-0.54	0.27
Std. Dev	0.59	0.51	0.51	0.59

(c) Bacon Island at Middle River

	Stage (ft)			
	HighHigh	LowHigh	LowLow	HighLow
Maximum	6.761	5.767	4.397	4.989
Minimum	1.325	0.687	-1.686	-1.368
Mean	3.16	2.27	-0.60	0.18
Median	3.139	2.181	-0.713	0.058
Std. Dev	0.61	0.52	0.53	0.62

(d) Bacon Island at Santa Fe Cut

	Stage (ft)			
	HighHigh	LowHigh	LowLow	HighLow
Maximum	6.826	5.81	5.003	5.003
Minimum	1.331	0.639	-1.714	-1.398
Mean	3.11	2.23	-0.65	0.12
Median	3.08	2.129	-0.768	0.004
Std. Dev	0.62	0.53	0.55	0.64

Table 2.3: Exceedance Probability and Corresponding Stages at Intake Site

Facility Location	Tidal Stage (ft)			
	90% Low-Low	10% High-High	10% Low-High	50% Low-High
Webb Tract, San Joaquin River	-1	3.8	2.75	2.1
Webb Tract, False River	-1	3.8	2.75	2.1
Bacon Island, Middle River	-1.1	3.9	2.9	2.2
Bacon Island, Santa Fe Cut	-1.1	3.9	2.9	2.2

2.4 Hydraulic Investigations

A number of hydraulic analyses were conducted to determine the overall layout of the integrated facilities. The objectives of the hydraulic analyses were to:

- determine the size and optimize the configuration of the integrated facility components while considering operational limitations and environmental constraints imposed on the project; and
- develop flow rating curves for each integrated facility showing the percentage of time the design flow can be met by gravity flow only, pumped flow only, or a combination of gravity and pumped flow.

Hydraulic analysis and design were completed on the following integrated facility components:

- Fish Screen Facility
- Gate Structures and Midbay
- Pumping Plant and Conduit Pipes
- Bypass Channel

2.5 Structural Design Investigations

State feasibility level structural analysis and design was prepared in sufficient detail to allow a feasibility-level cost estimate of the four proposed integrated facilities. In particular, structural analysis and design were completed for the structural components of the fish screen structure, the three gate structures, structures associated with the pumping stations and conduits, and for the sheet pile walls. Using the geological laboratory information provided by DWR, precast, prestressed concrete piles were designed for each structure such that settlement, cracking and tilting do not cause structural distress.

The *In-Delta Storage Program Integrated Facilities Structural Engineering Design and Analysis* (URS April 2003) report is provided in Appendix B. The report documents the design criteria, design basis and assumptions, design procedures, and results of the analysis. Drawings related to the structural components and foundations of the integrated facilities are provided in Appendix C.

Chapter 3: Fish Screen Facility

3.1 Introduction

The objective of the intake structure is to divert the required flow over the desired range of water levels in the channels and in the reservoir with hydraulic efficiency. The intake structure should also divert the design discharge under constraints imposed by operational and environmental considerations. Specifically, the intake structure should not hamper the movement of the fish species present in the river channel. The fish screens are the part of the intake structure intended to effectively protect juvenile fish from entrainment, impingement, and migration delay. The location of the fish screen facility is shown in Appendix C, Figure C.1.

3.2 Design Criteria

NMFS and DFG have established a number of criteria for the design and operation of fish screens installed at diversion points. The criteria are related to biological considerations, and hydraulics and hydrologic requirements for fish screening structures. A set of fish screen design guidelines that are applicable in the Delta were sent out for review. The following criteria were established based on review and suggestions from the In-Delta Storage Project Team as well as experts from State and federal fisheries agencies and DWR’s consultants. These criteria, listed below, will be used in designing the fish screen structures at the In-Delta Storage Project intake sites; two on Webb Tract and two on Bacon Island.

1. The screen should allow diversions up to 1500 cubic feet per second (cfs) at low stage and 2250 cfs at high stage.
2. The screen face shall be placed parallel to the river flow and adjacent bank lines. The intake facility should be designed to minimize or eliminate areas of reverse flow or slack water. These areas are predator habitat.
3. The structure must allow migrants to move freely in the channel adjacent to the screen area. The transition between the fish screen structure wing walls and the channel embankment should be smooth.
4. For self-cleaning screens and for all flow conditions, the approach velocity shall not exceed 0.2 ft/sec. The approach velocity is the water velocity 3 inches in front and perpendicular to the screen face.
5. The approach velocity in front of the screen should be distributed uniformly across the face of the screen.
6. The flow velocity component parallel to the screen face, known as sweeping velocity, must be twice the approach velocity (0.2 ft/sec).
7. NMFS recommends an upper limit of 60 seconds as the desirable fish passage time at approach velocities of 0.4 ft/sec. Fish passage time is defined as the length of time a fish is in front of the screen. For approach velocity of less than 0.4ft/sec, longer contact time may be applied with NMFS approval.
8. For vertical profile bar type fish screens, the screen openings should not exceed 0.0689 inches in width (1.75 mm).
9. The screen material shall provide a minimum of 27 percent open area.

10. For all hydrologic conditions, the screen material should be strong enough to withstand the water pressure caused by differential head over the screen faces. The fish screen material used should be corrosion resistant and antifouling.

11. The head difference to trigger fish screen cleaning shall be a maximum of 0.1 feet. To avoid flow impedance and violations of approach velocity criteria, a cleaning frequency of 5 minutes is desired.

12. Structural features shall be provided to protect the fish screens from large debris.

3.3 Intake Site Water Levels

3.3.1 Maximum Stage

The fish screen facility will be designed for a 100-year return period maximum river stage. To account for climate change effects, the system performance will be checked with a 300-year return period flood. The design flood levels and their respective return periods were taken from the *In-Delta Storage Program Flooding Analysis*, (URS April 2003) and are summarized in Table 3.1.

Table 3.1: Design Flood Levels

Facility	Flood Stage (ft)	
	100-year	300-year
Webb Tract, SJR	7.0	7.2
Webb Tract, False River	7.0	7.2
Bacon Island, Middle River	7.2	7.5
Bacon Island, Santa Fe Cut	7.3	7.5

3.3.2 Tidal Stage

The diversion capacity of each fish screen facility will be 1500 cfs for low stage operations and 2250 cfs for high stage operations. The facility will be operated year-round and it will be able to deliver the design discharge at low flow as well as high flow conditions. The low stage condition is defined as the tidal stage when the river stage is at 90 percent level of the Low-Low tidal stage. Likewise, the high stage condition is defined as the tidal condition when the river stage is at or above the 50 percent level of the Low-High tidal stage. The probability levels and the corresponding stages are summarized in Table 2.3. These two tidal stages will be used to determine screen height, total width of the fish screen facility, and top level of the steel face wall.

3.4 Design and Layout

3.4.1 General Layout

The fish screens will be placed in a location where the river alignment is fairly straight such that the fish screen face is nearly parallel to the adjacent riverbank. This will minimize the contact time of the migrating fish species with the fish screen facility. The screen length will be sized such that it can deliver the design discharge of 1500 cfs during low flow conditions and 2250 cfs during high flow conditions in the slough. To protect the migrating juvenile fish the approach velocity should be at or below the 0.2 feet per second criterion. To ensure proper functioning of the units, the velocity distribution in front of the screen should be uniform. Each facility is located in areas influenced by tides, and depending upon the tidal cycle and hydrology the channel velocity will fluctuate and may even change its direction. Therefore, the criteria dealing with the sweeping velocity and passage time requirements may be applicable to the screens for flows in two tidal directions.

The components of each fish screen facility will include a log boom, fish screen, cleaning device, adjustable baffles, debris collection and removal system, reinforced concrete box culvert structural section, and an access road. The fish screen structure will be placed at the average topographic elevation of each site location as shown in Table 3.2. The layout plan and cross section of the proposed fish screen facility are shown in Appendix C, Figures C.5 and C.6.

3.4.2 Fish Screen Sill Elevation

The fish screen sill elevation is based on the site topography and nature of site soils. Table 3.2 summarizes the selected sill level for each site, which is slightly below the average topographic level measured at each location. Lowering the sill level provides additional screen height and reduces the overall width of the facility. Further lowering of the sill level would provide additional screen height but it may exacerbate the accumulation of sediments in the fish screen facility.

Table 3.2: Intake Site Topographic and Sill Elevations

Facility	Average Topographic Elevation	Sill Elevation	Top of Levee Elevation (ft)
Webb Tract, SJR	-10	-12	11.0
Webb Tract, False River	-13	-15	11.0
Bacon Island, Middle River	-10	-12	10.2
Bacon Island, Santa Fe Cut	-7	-9	10.4

3.4.3 Fish Screen Facility Width

The width of the intake facility should be sufficient to pass the 1500 cfs design flow during low flow conditions in the river channel as well as to pass the 2250 cfs design flow during high flow conditions in the river channel. The required screen width for each of these flow conditions will be determined as

$$w = \frac{Q}{hv} \qquad \text{(Equation 3.1)}$$

where w is the width of screen required, h is the screen height, v is the limiting approach velocity in front of the screen, and Q is the design flow. As described earlier, the limiting approach velocity for this project is 0.2 feet per second.

For the design flow of 1500 cfs, the minimum gross wetted screen area required is 7500 ft^2. The intake facility is designed to deliver this flow at minimum slough stage, which is the 90 percent probability of exceedance level for the low-low tidal stage. The difference between the 90 percent low-low tidal stage and the fish screen sill level gives the effective fish screen height. Using this height, the limiting approach velocity, and a design flow of 1500 cfs, the required fish screen width can be determined from Equation 3.1. This procedure is repeated for each site.

For the design flow of 2250 cfs, the minimum gross wetted screen area required is 11,250 square feet. The intake facility is designed to deliver this flow when the slough stage is higher than the 50 percent probability of exceedance level of the low-high tidal stage. Following the same procedure described above, but for a screen height relative to the 50 percent exceedance level of the low-high tidal stage, the required fish screen width can be determined from Equation 3.1. This procedure is repeated for each site.

Once the required screen width for each site is determined under both flow conditions, as described above, the results are compared and the larger of the two screen widths is chosen for the design. This procedure is repeated for each site and the desired screen widths are summarized in Table 3.3. Depending on the slough stage conditions, which vary throughout the Delta, the height of the fish screen will vary at different site locations.

For all sites, the required width of screen is too large for a single screen. Based on structural considerations, the required screen width will be divided into smaller bays. Each bay will be separated by a 2-foot wide concrete pier and each bay will have a clear span of 20 feet. In addition to separating the bays, the piers will support the fish screens, stop logs, adjustable baffles, mechanical equipment and deck slab. The piers will extend from the screen face a distance equal to the width of the deck required for cleaning, debris removal, and operator access. The total number of bays required, along with the total width of each facility and the screen top elevations are given in Table 3.4.

Table 3.3: Fish Screen Widths Based on Permissible Approach Velocity

Facility	Fish Screen Sill Elevation	WSE (%90 LL) (ft)	Clear Screen Opening (ft)	Width of Screen Required (ft)
Webb Tract, SJR	-12	-1	11	800
Webb Tract, False River	-15	-1	14	660
Bacon Island, Middle River	-12	-1.1	10.9	800
Bacon Island, SF Cut	-9	-1.1	10.9	1020

Table 3.4: Number of Bays, Total Width and Screen Top Elevations

Facility	Number of Bays	Total Width of Intake Facility (ft)	Top of Screen Elevation (ft)
Webb Tract, SJR	40	878	2.49
Webb Tract, False River	33	724	2.39
Bacon Island, Middle River	40	878	2.49
Bacon Island, SF Cut	51	1120	2.59

3.4.4 Screen Type and Layout

The fish protection system for all intake sites will be passive screen. The screen will be vertical profile bar as shown in Figure 3.1 and the clear opening between vertical bars will not exceed 0.0689 inches (1.75 mm). The screen will have a minimum screen area of 50 percent of the gross wetted screen area. The vertical bars will be configured such that the flatter side of the bar faces upstream, which will reduce the chances of the panel becoming clogged. To minimize the fouling effect, the screen will be made of type 304 stainless steel.

Each bay will have two screen panels installed side-by-side across the bay. The panels will be separated by a guide rail channel section placed at the center. Therefore, each screen will be 10 feet wide. This smaller width of the fish screen panels will reduce the possibility of excessive bending but it will increase the project cost. For design purposes, a 2-foot head difference across the screen face will be assumed. At any time, the total bending (deflection) will be kept at or below 1/8 of an inch. Excessive bending of the screen panels will hamper the cleaning operation, eventually causing the screen to fail.

Screen guides will be provided for removing and reinstalling the panels for inspections and maintenance. The vertical profile bars will be supported at intermediate points by the channel sections to minimize deflection. The top and bottom of the screen will be strengthened by a screen panel structure frame. Lifting eye cutouts will be provided in the screen frame to facilitate removal of the screen panel. The screens will be kept in place by gravity and they will move along the screen guides. The top of the screen will be sealed and all of the openings above the screen should be smaller than the recommended size of the screen openings which is 0.0689 inches (1.75 mm). The top of the screen will have a transition plate that will allow the screen cleaning system to move smoothly from the screen face to the steel face wall (see Section 3.4.6).

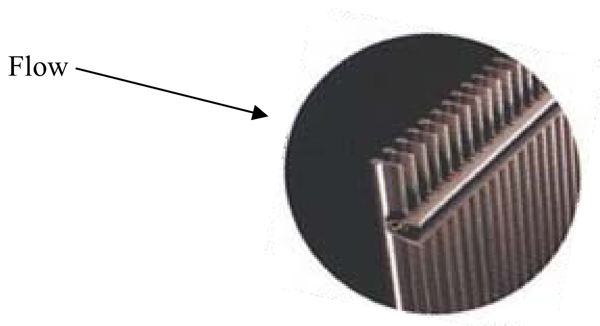


Figure 3.1 – Vertical Profile Bar Screen Manufactured by Hendrick Screen Company, Inc.

3.4.5 Screen Top

The top of the screen will extend to a minimum elevation of the 50 percent probability of exceedance for the low-high tide level. The low-low and high-low tides will exceed this stage less than 5 percent of the time (see Appendix A, Figures A.1 through A.4 for a graphical representation of the tidal analysis). This is to ensure that the top of the screen will extend to a level that the lifting eyes become visible for a portion of time during most days. This screen height will allow regular maintenance and repair activities to be performed during the low tide periods. The actual top of screen elevation will be rounded to an increment suitable to accommodate manufacturing requirements. The top of screen elevations are given in Table 3.4.

3.4.6 Steel Face Wall

From the top of the fish screen to the top of the intake structure deck (engineered embankment elevation), a steel face wall will be constructed of 1¼-inch thick steel plates. The steel face wall will prevent excess flows from passing through the facility above the fish screen when the river is at higher stages and will also protect the deck slab from wave run-ups. The steel face wall will be placed in the groove extending all the way to the sill level and will be aligned such that there is a smooth transition for the fish screen to move up and down. This will allow the steel face wall to be used as a stop log (by lowering it to the sill), preventing water from entering the fish screen facility when maintenance and inspections are required. Stop log slots will also be provided at the downstream end of the concrete piers as discussed in Section 3.4.13. The interface between the face wall and fish screen will be sealed such that the resulting openings are smaller than the allowable fish screen opening.

3.4.7 Log Boom

The structure should be able to exclude the flow of undesired materials, such as debris, into the screen. Installation of trash racks required the extension of the piers beyond the fish screen. Extending of the pier will create pockets of slow moving water in front of the screen, which is not a desirable feature.

To prevent the flow of debris coming into the screen a floating log boom will be provided. The floating log boom will be supported by a series of dolphin piles driven in the channel bed. The floating log boom will be equipped with a suspended debris fence that will prevent the debris from reaching the screen face. The suspenders will be made of non-corrosive steel and will be placed 6-inches apart. To assure proper functioning of the log boom, the bottom of the hanging chain should be stiff for all flow levels. This will help prevent Hyacinth (a common weed found in Delta channels) from floating past the log boom and clogging the fish screen. The log boom fence will be inspected and cleaned regularly by divers.

3.4.8 Cleaning Device and Frequency

The head difference to trigger fish screen cleaning shall be a maximum of 0.1 feet. To avoid flow impedance and violations of approach velocity criteria, a cleaning frequency of five minutes is desired. However, for the fish screen facility located at Old River and operated by the Contra Costa Water District, the recommended cleaning frequency is 20 to 40 minutes. Contingent upon DFG/NMFS/USFWS approval, the time lags between the cleaning periods could be higher than the recommended time of 5 minutes.

At each site, three to four cleaning units on single rail systems will be installed. The number of units is based on the overall length of the fish screen. The number of units could change during the detailed design depending upon the manufacturer's recommendations and regulatory agency approval. In case there is more than one cleaning device, the cleaning devices will start from opposite ends of the intake facility. The cleaning device will be a single stroke vertical scraping type. The constant cleaning will maintain a constant velocity across the screen face. A typical cleaner manufactured by the Atlas Polar company is shown in Figure 3.2. The cleaning device will be manufactured by Kuenz America (Type TRCM E 35), Atlas Polar Cleaning Systems (Type ST8100 Hydro-brush) or from some other established manufacturer. A manufacturer will be selected based upon the performance reliability of the installed devices by that manufacturer.

Figure 3.2 – Vertical Strike cleaning brush Manufactured by Atlas Polar Hydrobrush Cleaning System

3.4.9 Debris Removal System

Once the debris has been pulled out from the screen it must be properly collected and transported to the disposal site. Thus, each cleaning brush will be accompanied with a collecting dumpster. These dumpsters, when filled, will be transported to the disposal site.

The fish screen, cleaning device and debris removal systems are an integrated system. It is preferred, then, that the entire system be fabricated by one manufacturer. If, however, different manufacturers supply various components of the screening facility, close coordination shall be ensured. The selection of the manufacturer will be decided based upon the reliability of performance of the installed devices by the manufacturer.

3.4.10 Intake Structure and Deck

A reinforced concrete box culvert section was determined to be most appropriate for the fish screen structure. Structural design details are presented in Appendix B. The top slab of the box culvert structure will act as a bridge deck along the top of the intake facility. The concrete deck will be wide enough to accommodate the cleaning rails, walkway, debris removal system and fish screen handling cranes. Lighting and safety railings will be provided along the deck to allow operation of the facility 24 hours a day. The deck and the levee top will have the same elevation as given in Table 3.2. A summary of the fish screen specifications for all sites is presented in Table 3.5.

Table 3.5: Summary of Fish Screen Specifications

Item	Required Specification
Required Gross Wetted Area	7,500 sq.ft. during low stage 11,250 sq.ft during high stage
Screen Open Area	50 percent of Gross Wetted Area
Screen Type	Vertical Profile Bar
Screen Opening	0.0689 inches (1.75 mm)
Length of Screen	15, 18, 15, and 12 ft *
Screen Size	Screen Length x 10 ft
Permissible Bending	1/8 inch
Screen Material	Type 304 Stainless Steel
Side Support	Steel Channel Guide
Vertical Support and Removal	Gravity and through lifting eyes
Cleaning Type	Single Stroke Vertical Scraper
Number of Cleaning Units	3, 3, 3, 4 *
Debris Removal	Continuous Type

* Screen lengths and number of cleaning units are for Webb Tract @ SJR, Webb Tract @ False River, Bacon Island @ Middle River and Bacon Island @ SF Cut, respectively.

3.4.11 Foundation

To minimize the effects of differential settlement resulting from static loads and the effects of dynamic loads, a pile foundation system has been considered in this study for supporting the fish screen structure. As detailed in Appendix B, 14-inch square precast, prestressed piles with an allowable capacity of 45 tons were selected for preliminary design purposes.

3.4.12 Sediment Handling and Removal

To reduce sedimentation, the bottom of the intake channel will be sloped towards the river, as indicated in Appendix C, Figure C.6. The sediments that are carried through and deposited inside the fish screen may be flushed out from behind the screens by using high pressure water jets through pipes installed within the base slab and concrete sill or by another suitable method. The most suitable sediment removal system will be determined during final design. The flushed-out (or re-suspended) sediments will then be carried downstream of the fish screen structure and deposited in the Transition Pool. The Transition Pool is the area between the fish screen facility and Gate #1, as shown in Appendix C, Figure C.1, and will be dredged as needed to remove accumulated sediments.

3.4.13 Stop Log Guides and Adjustable Baffles

Stop log slots will be provided at the downstream end of the concrete piers. The slots will be used to place the stop logs to prevent water from entering the fish screen facility from the downstream end. As mentioned in Section 3.4.6, the steel face wall can also be used as a stop log to prevent water from entering the fish screen facility from the upstream end. With all stop logs and steel face wall lowered, maintenance and inspections can be performed for each bay individually.

A second set of slots will also be provided at the downstream end of the concrete piers for the installation of adjustable baffles. These adjustable baffles will help provide uniform flow through the entire width of the fish screen facility.

3.4.14 Scour and Erosion Protection

The channel bed upstream and downstream of the fish screen structure will be provided with a riprap blanket for protection against scouring and erosion. On the downstream side (behind the screen) where the flow velocity is expected to be low, the riprap blanket will be extended laterally to a distance of 10 feet beyond the edge of the fish screen piers. On the upstream side, the riprap blanket will be between 10- to 20-feet long from the beginning of the fish screen slab projecting out toward the river channel. Regular maintenance will be undertaken to ensure the integrity of the riprap blanket for providing the desired level of protection against scouring and erosion. Figure C.6 in Appendix C illustrates the proposed configuration of the riprap blanket.

3.5 Seismic Considerations

During a large earthquake the system will be designed such that screen system fails before the concrete deck. This is because it is more economical to replace the screen systems rather than the concrete deck.

3.6 Hydraulic Considerations

The possibility of flow reversal from the fish screen should be minimized. This will be accomplished by providing gradual transitions (both vertical and horizontal) of the abutments, side embankments, and intake floor surfaces. The adjustable baffles will also help to minimize flow reversal.

The head loss in the structure should be minimized and there should be no flow separation. The gated structure downstream of the fish screen (Gate #1) will be aligned with the center of the fish screen structure. Any possibilities of vortex formation in the transition area downstream of the fish screen (between the fish screen and Gate #1) should be minimized.

Chapter 4: Gate Structures and Midbay

4.1 Introduction

The objective of the gate structures and midbay area is to control the flow, under varied slough and reservoir stages, into and out of the reservoir with hydraulic efficiency. As controlling and regulating structures, design considerations include structural integrity, cost efficiency and reliability of operation. This chapter describes the design criteria, general layout, and hydraulic design of the gate structures and the midbay area of the integrated facility.

4.2 Design Criteria

Hydraulic design criteria for the gate structures and midbay area are listed below.

1. Velocity in unlined sections should not exceed 3 ft/sec. This will prevent scouring.
2. Energy dissipation structures or stilling basins should be provided to prevent damage to the sections of the integrated facility downstream of point of control.

4.3 General Layout

Each integrated facility consists of three gate structures and one midbay area; their general layouts are shown in Appendix C, Figures C.1 through C.4. The primary purpose of the gate structures is to regulate the flow of water into and out of the reservoir through the integrated facility. Each gate structure, however, serves a unique purpose in the reservoir operations. Gate #1 is used for inflow (diversions) only, Gate #2 is used for both inflow and outflow (releases), and Gate #3 is used for outflow only. All three gates will be vertical lift slide gates that can be regulated mechanically or manually.

The total number of gate structures required in the facility was selected to maximize the use of gravity flow. In particular, Gate #3 was added to achieve maximum gravity flow releases under the year-round reservoir operations as modeled in the CALSIM-II daily model.

The midbay area serves as a transition pool for all three gates. Flow into and out of each gate structure is directed using smooth and straight transitions to minimize hydraulic losses and cavitation potential. Simple convergence transitions with vertical sidewalls will be used. The lengths of transitions will be designed to prevent flow separation.

Energy dissipation devices will be used to dissipate excess energy at the downstream end of each gate structure. The downstream end of a gate structure depends on the direction of flow through the gate. As described in Section 1.2.3, the regulation of flow during diversion and release operations will be achieved through the combined operations of the three gate structures, pumping plant, and conduit pipes. Gate #1 has only one energy dissipater on the downstream side of the gate sill since it is used strictly during diversion operations to regulate flows into the midbay and will be closed during release operations. Gate #3 is used strictly during release operations to regulate flows from the midbay into the bypass channel and has only one energy dissipater which is located between the gate sill and the bypass channel. Gate #2 is used during both diversion and release operations and has energy dissipaters on both sides of the gates. The energy dissipaters for the gate structures are discussed further in Section 4.4.2.

4.3.1 Gate Location and Sill Levels

The gate structures are centered along the sides of the midbay. This positioning provides for uniform flow as water approaches the gate structures. Smooth transitions and uniform flow will lessen the head loss as water passes through the gates.

The sill levels for Gate #1 and Gate #3 were determined based the existing ground elevations and slough bed levels at each integrated facility location. The sill level of Gate #2 was selected to achieve the desired level of reservoir emptying. Table 4.1 summarizes the selected sill elevations for each gate structure. A metal plate should be provided at the top of each gate sill such that the underlying concrete is not damaged during repeated opening and closing of the gates.

Table 4.1: Gate Structure Sill Elevations

Item	Integrated Facility Elevation (ft)			
	Webb Tract, San Joaquin River	Webb Tract, False River	Bacon Island, Middle River	Bacon Island, Santa Fe River
Gate #1	-12	-15	-13	-8
Gate #2	-18	-18	-16	-16
Gate #3	-15	-16	-12	-8

4.3.2 Midbay Floor Level

The midbay floor should be deep enough to empty the reservoir to the desired level and the midbay water depth should be sufficient to meet submergence requirements of the pumps. The midbay floor should also be deep enough for flow through the gate structures to form a hydraulic jump and to dissipate energy within the pool. Calculations for the hydraulic tail water depth requirements are given in Appendix A.2. The energy dissipaters for the gate structures are discussed in Section 4.4.2.

The bottom and sides of the midbay will be covered with riprap. The transition pool upstream of Gate #1 will retain most sediments and sediments that accumulate in the midbay will be removed by periodic flushing and cleaning. Water level variations in the midbay will be controlled by gate and pumping operations to be minimum and gradual. Table 4.2 summarizes the adopted values of the midbay floor elevations and minimum required midbay water levels during diversions.

Table 4.2: Midbay Floor Elevations and Minimum Required Water Levels During Diversion

	Integrated Facility			
	Webb Tract, SJR River	Webb Tract, False River	Bacon Island, Middle River	Bacon Island, Santa Fe Cut
Midbay Floor Elevation (ft)	-24	-24	-22	-22
Minimum Recommended Water Level in Midbay (ft)	-14	-17	-15	-10

4.4 Hydraulic Design

4.4.1 Gate Selection

Gate structures #1 and #2 each have three 12-foot wide by 10-foot high gate panels separated by 2-foot wide piers. Gate structure #3 has two 12-foot wide by 8-foot high gate panels. The number of gate panels and the maximum gate openings were fixed to maximize gravity flow through the gates and are shown in Table 4.3. Gate design procedures are summarized in Appendix A.4. All of the gates will be vertical-type mechanically driven painted steel roller gates with hydraulic cylinder actuators, however, in case of power failure, they will be equipped for manual operation. Channel sections will be embedded in the piers so that the gates are held in position and can be moved up and down. Layout plans and other details of the gates are shown in Appendix C, Figures C.7 through C.12.

Table 4.3: Number of Gate Panels, Gate Width, and Gate Height

Gate No.	Number of Gate Panels	Gate Panel Width (ft)	Maximum Gate Opening (ft)
Gate #1	3	12	10
Gate #2	3	12	10
Gate #3	2	12	8

4.4.2 Energy Dissipation

Gate #1 Energy Dissipaters

A very high head differential is possible at Gate #1 when slough levels are relatively high compared to reservoir levels. In this case, a large amount of energy must be dissipated in the midbay just downstream of Gate #1. This energy can be dissipated by a submerged jump downstream of Gate #1 provided a minimum depth of water is available in the midbay.

The methodologies used in determining the water surface profile downstream of Gate #1 are described below and details are given in Appendix A.2.

Water surface profiles were generated first by assuming an empty midbay level and maximum flow on the river side and then by computing the minimum tail water depth required to dissipate the energy through a submerged hydraulic jump. Minimum tail water depth is the sequent depth d_2, corresponding to the depth d_1 of water at the end of the sloping apron, which extends from Gate#1 to the midbay floor. The depth d_1 and velocity were used to compute the Froude Number of the flow entering the jump. The Froude Number for all integrated facilities is in the range of 4.5 to 4.6, which suggests a steady hydraulic jump. An S2 profile was developed and combined with the minimum tail water depth to generate the final water surface profiles shown in Appendix A, Figures A.5 through A.8.

The required midbay floor elevation was determined such that pump submergence requirements are met. This elevation, however, does not provide adequate sequent depth required to form a hydraulic jump. Energy dissipation at Gate #1 will be handled through constrained operations of the system. A minimum tail water depth in the midbay must be established prior to filling the reservoir. To achieve this Gate #1 should be partially opened when the tide level is low and Gate #2 should be closed. Alternatively, a sump pump can be used to fill the midbay to achieve the required minimum tail water depth. Once the minimum tail water depth has been reached Gate #1 and Gate #2 can be fully operated. The recommended minimum tail water depths are summarized in Table 4.2.

Gate #2 Energy Dissipaters

As previously mentioned, Gate #2 is designed as a two-way hydraulic structure connecting the midbay to the reservoir. Being a two-way structure, Gate #2 has energy dissipaters on both sides of the gates.

The floor on the reservoir side is used to dissipate the energy during the diversion of water into the reservoir. The reservoir may be empty during some diversion periods, indicating that there is no minimum tail water depth available on the downstream side of the gate to dissipate excess energy through a hydraulic jump. Because of this, the floor on the reservoir side of Gate #2 is designed to dissipate energy even without adequate tail water depth. Starting from Gate #2, the floor will expand at a 45-degree transition angle. This gradual expansion of the wing walls will help to avoid flow separation and cavitation problems. The horizontal concrete slab extending from the gate sill to the reservoir floor is about 52 feet long and will have an end sill followed by 10 to 20 feet of riprap protection. As the flow passes through the transition, its velocity will be reduced to the permissible limit of 3 ft/sec.

The concrete floor is followed by a dredged and graded outlet channel, which extends to the lower elevations of the reservoir to allow for maximum drainage of the reservoir. To minimize seepage pressures, a concrete cutoff wall will be provided at the end of the concrete structure.

During the release of water from the reservoir to the midbay, the energy dissipation downstream of Gate #2 will be achieved with a submerged tail water depth procedure as described for Gate #1.

Gate #3 Energy Dissipaters

The outlet extending from the sill of Gate #3 into the bypass channel will consist of a flared concrete transition. This outlet transition, along with sufficient tail water depth provided by the slough level, will dissipate the energy and, when the water reaches the bypass channel, the velocity will be within the permissible limit of 3 ft/sec. The outlet will be flared at a 45-degree transition angle such that flow separation is minimized. The length of the concrete floor is about 52 feet long and will be provided with a concrete end sill and cutoff walls at both ends to reduce seepage pressures.

4.4.3 Flow Rating Curves

Flow rating curves were developed at all integrated facility locations for Gate #1 (inflow only), Gate #2 (inflow only), and Gate #3 (outflow only). Each rating curve shows the percentage of time the design flow can be met by gravity flow only, pumped flow only, or a combination of gravity and pumped flow. Each rating curve also shows the corresponding total head required between the reservoir and the slough. Rating curves were developed for both diversion and release operations. Information from the DSM2 and CALSIM-II computer models and gate geometry was combined to develop the curves. DSM2 Hydro provided hourly slough stage data at each integrated facility location, whereas CALSIM-II provided reservoir stage, and inflow and outflow data on a daily basis.

The following procedure was used to develop inflow-rating curves for diversion operations. For each hour of the day, maximum gravity flow diversions were calculated based on the difference between slough and reservoir stages and head losses. Total daily gravity flow diversions were compared with total daily diversion requirements to determine the need for pumping. If the total daily gravity flow was not sufficient to meet the daily diversion requirement of CALSIM-II, pumping was triggered. This method was used to prepare the outflow rating curves for release operations as well. Inflow and outflow flow rating curves for each gate at all facility locations are given in Appendix A, Figures A.9 through A.20.

4.5 Miscellaneous Design Features

4.5.1 Face Wall and Deck Level

A concrete face wall will be provided from the top of the gate opening to the deck slab level. The area between the concrete face wall and the gate will be properly sealed. It will also provide a smooth surface to prevent leakage when the gates are closed. The top of the deck of each gate structure will have a 35-foot wide roadway. For easy access of cranes and other service equipment, the deck elevation will be at the same elevation as the adjacent embankment. The deck top will also provide areas for installing the gates and hoist mechanisms. The gates will be equipped for remote automatic operation and they can also be operated manually when needed.

4.5.2 Mechanical Components

It was assumed that there will be times when Gates 1 and 2 will be opened with a large differential head. Furthermore, it was assumed that the gates will be used to throttle the flow during filling of the midbay. For these reasons, a cable hoist was ruled out and hydraulic cylinder actuators were chosen to operate the mechanically driven painted steel roller gates. Although there may be no need to throttle the flow at Gate #3, hydraulic cylinder actuators were chosen for consistency and ease of maintenance. Also for ease of maintenance, the hydraulic cylinders will be located high enough to be out of the water. Given that there are three gates each at gate structures 1 and 2, a single hydraulic power unit can be provided to operate the gates at each of the three gate structures. If one of the HPUs failed, flow can be prevented by the combination of the other two gate structures; therefore, backup HPUs are not necessary. The gates will be opened and closed frequently. If electric actuators were used, the frequent actuation would probably cause excessive wear on the self-locking gears.

4.5.3 Trash Racks

To stop the flow of debris and floating particles, trash racks will be provided at the beginning of Gate #2. The trash racks will be made of anti-fouling steel and the clear spacing between the bars should not be more than 2 inches. This spacing requirement will prevent adult/predator-sized fish from exiting the reservoir. The trash rack bars and supports will be designed for a minimum of 25 percent of the reservoir head to which they would be subjected if completely clogged. For easy placement and removal, the trash racks at Gate #2 will be placed in slots or guides. The racks will remain in place by gravity action and hence no additional anchorage is needed.

A trash rack is not necessary immediately in front of Gate #1 because the fish screen will prevent any debris from entering the transition pool in front of Gate #1. Gate #3 does not require a trash rack either because debris is not expected to get into the midbay.

4.5.4 Stop Logs

In addition to normal operating (service) gates, stop log slots will be provided at each gate. These slots will be used to place stop logs to stop the flow of water, so maintenance and inspections of the gate, gate slides, or gate sill areas can be carried out. Each stop log will be made out of aluminum panels.

4.5.5 Sedimentation Control

The inlet/outlet gates will be protected against suspended silt load. The transition pool upstream of Gate #1, having very low velocities, will act as a settling basin to trap sediments before they enter the reservoir during diversion operations. The accumulated sediments in the transition pool will be removed by mechanical means. The midbay is not expected to retain sediments because frequent flushing and cleaning will be conducted in this area.

4.6 Outlet Channel

A dredged and graded trapezoidal outlet channel will extend from Gate #2 to the lower elevations of the reservoir to allow for maximum drainage of the reservoir. The configuration and extent of this channel will be determined during final design of the project.

Chapter 5: Pumping Plant and Conduit Pipes

5.1 Introduction

The pumping plant is located adjacent to the midbay on the side opposite to Gate #1 and the conduit pipes stretch from the reservoir side of the integrated facility to the bypass channel, as shown in Appendix C, Figures C.13 through C.19, and C.28. The pumping plant serves to:

- supplement diversion and release gravity flows when sufficient head is not available to meet the desired flow rate, and
- meet the entire flow rate when no head (or negative head) is available.

The pumping plant consists of five vertical-type pumping units (three 400 cfs and two 150 cfs units), totaling a maximum pumping capacity of 1500 cfs. The smaller pumps, having lower submergence requirements than the larger pumps, can be used to pump water out of the reservoir at lower elevations, allowing flexibility in operations when needed.

The conduit pipes will be used to discharge water into the reservoir and bypass channel during diversion and release operations, respectively. For both operational conditions, the flow direction is controlled by two butterfly valves installed in each conduit pipe, as shown in Appendix C, Figure C.28. For diversions to be made through the pumping plant, the valves closest to the bypass channel will be closed and the valves closest to the reservoir will be open. The opposite is true for releases. The conduit pipes can also be used for gravity flow releases to supplement the gravity flow releases through Gate #3. This can be achieved by opening both butterfly valves in each conduit pipe.

The pumping plant layout, design and layout of the conduit pipes and selection of mechanical and electrical equipment is discussed in more detail throughout this chapter.

5.2 Design Criteria

5.2.1 Pumping Plant Design Criteria

The following design criteria will be applied in the hydraulic design of the pumping plant:

1) The pumping plant shall supply water under the following cases:

   a) Diversions

      i) Pumping only: for diversions into the reservoir when the reservoir level is the same as or higher than the river level.

      ii) Combination (pumping and gravity): for diversions into the reservoir when the reservoir level is lower than the river level, but gravity flow is not enough to achieve the desired level of diversions.

   b) Releases

      i) Pumping only: for releases from the reservoir when the reservoir level is the same as or lower than the river level.

ii) Combination (pumping and gravity): for releases from the reservoir when the reservoir level is higher than the river level, but gravity flow is not enough to achieve the desired level of releases.

2) Pumping unit submergence requirements shall be met at all times during pumping operations. This will help to prevent cavitation of the pumps.
3) The inlet from the midbay to each pumping unit shall provide a smooth transition to minimize head loss. A formed suction intake shall be used to ensure a smooth transition.
4) The intake basin shall be configured to avoid vortex formation in the midbay and to minimize flow separation.
5) The pumping station shall be designed to allow for maximum drainage of the reservoir.
6) The midbay shall be completely drained for maintenance operations. This will require the design and installation of a smaller sump pump and discharge conduit.
7) The forebay and afterbay water surface elevations for the proposed pumping plant are given in Table 5.1.

Table 5.1: Pumping Plant Forebay and Afterbay Water Surface Elevations

Forebay and After Bay Water Surface Elevation	Integrated Facility Location							
	Webb Tract San Joaquin River		Webb Tract False River		Bacon Island Middle River		Bacon Island Santa Fe Cut	
	River to Reservoir	Reservoir to River	River to Reservoir	Reservoir to River	River to Reservoir	Reservoir to River	River to Reservoir	Reservoir to River
Maximum	6.8	4	6.4	4	6.8	4	6.8	4
Normal	-1		-1		-1.1		-1.1	
Minimum	-1.7	-18	-1.5	-18	-1.7	-16	-1.7	-16

5.2.2 Conduit Pipe Design Criteria

The following design criteria will be applied in the design of the conduit:

1) The conduit pipes shall be designed to flow under pressure and shall be sufficient to pass a total flow rate of 1500 cfs.
2) The conduit pipes shall have a minimum slope of 1-foot per 1000-feet to allow for drainage. Where conduit pipe sections have no slope, a drainage system shall be installed.
3) A cathodic protection system shall be considered in combination with protective coatings to ensure adequate protection and longevity of conduit, pumping units, gates, valves and other appurtenances.
4) A trash rack shall be provided at the reservoir side of the intake/discharge conduit.
5) All concrete conduit pipes should be manufactured in accordance with ASTM C76M specifications.

5.3 Pumping Plant and Conduit Layout

The pumping plant consists of three 1500 hp pumps with a capacity of 400 cubic feet per second (cfs) each and two 800 hp pumps with a capacity of 150 cfs each, totaling five pumping units with a maximum pumping capacity of 1500 cfs. This combination of pump units was selected based on the proposed year-round operation of the facilities. All pumping units will be vertical-type mixed flow pumps driven by a motor connected to a right angle gear. A formed suction intake (FSI) will be mounted to each pump below the impeller to eliminate vortex formation in front of the pump. The FSI reduces the submergence depth requirements, which makes it is possible to operate the pumps when the reservoir water levels are low. More details on pump selection are provided in Section 5.4.2.

Stop log slots will be provided in front of each pumping plant intake. This will allow individual pumping units to be shut down and serviced while the rest of the units continue operating. A gantry crane will also be provided to facilitate required maintenance and inspections of the pumps, valves, motors and gears. Layout and details of pumping plants and conduits are shown in Appendix C, Figures C.13 through C.33.

5.3.1 Plant Superstructure

The pumping plant superstructure will consist of three levels, an upper level, middle level and lower level. The upper level will support the right angle gears, pump motors, and gantry crane, and will have covered openings that provide access to the pump units and the discharge valves. The upper level will not be housed (enclosed) by a supported structure, in other words, it will be exposed to the elements. The switch yard will be located on the top of the embankment adjacent to the upper level. The middle level will support the pump units, pump discharge pipe, butterfly valves (and associated floor mounted hydraulic actuators) and a hydraulic power unit (HPU). The middle level will also house the motor control room and office. The lower level will house and provide access to the formed suction intakes. Two 60 gpm (½-hp) sump pumps will also be provided in the lower level of the pumping plant.

The pumping plant will also have a service elevator next to the motor control room and stairs at both ends of the superstructure, providing access to all three levels.

5.3.2 Piping Layout

Figures C.13 through C.19 in Appendix C, show the piping layout for the entire pumping plant and conduit pipes including the pump discharge pipe connections to the conduit pipes. The drawings include the discharge pipe for all five pumps, the three conduit pipes, couplings, butterfly valves, and hydraulic actuators.

The conduit pipes consist of two eight-foot diameter pipes and one six-foot diameter pipe. Pump Unit No. 1 will split its discharge between the two 8-foot conduit pipes, with half of its flow going to each pipe. Pump Unit No. 2 will discharge into one of the 8-foot conduit pipes and Pump Unit No. 3 will discharge into the other. Both 150 cfs pumps (Pump Units No. 4 and No. 5) will discharge into the 6-foot conduit pipe; however, because of the low head that may occur when only one of the 150 cfs pump units is running, both pumps may have to be operated when the pumping head is low. A summary of conduit pipe design is provided in Section 5.3.3.

There are two butterfly valves installed in each conduit pipe and the direction of flow through the each pipe is controlled by the joint operation of the two butterfly valves. As shown in Appendix C, Figure C.28, the butterfly valves in each conduit pipe are aligned with one another and are housed in a valve vault. Each valve vault contains a hydraulic power unit to operate the floor-mounted hydraulic actuators, which open and close the butterfly valves. A spacing of six pipe diameters (discharge pipe diameters) was chosen as the distance between the pump and the discharge valve to minimize the probability of premature valve failure due to turbulence.

The conduit pipes slope downward from the ends toward the valve vaults to allow for drainage. A 60 gpm (½-hp) conduit drain pump and valve are also provided in each valve vault. Smaller piping will be connected to each conduit pipe and the drain pump, which will allow the conduit pipes to be completely drained.

5.3.3 Conduit Pipe Design

Pipe Selection

The conduit pipes were designed to carry a combined design discharge of 1500 cfs under a maximum permissible velocity of 12 ft/sec. A variety of pipe sizes, configurations, and materials were considered to optimize the pipe sizes for various hydrologic and operating conditions. The chosen configuration consists of one 6-foot and two 8-foot diameter pipes. The configuration of the pump discharge pipe connections to the conduit pipes is described in Section 5.3.2. Considering their size, strength and economy, reinforced concrete pipes are being recommended for all integrated facility locations.

Given the variation of available head between the reservoir and the river, gravity flow capacity through the conduit pipes was determined by the energy balance approach. The capacity calculations include pipe friction losses and minor head losses (such as entrance, exit, valves, fittings, contractions, expansions, etc.). Hydraulic design procedures are given in Appendix A.5. The flow rating curves that include gravity flow releases through the conduit pipes are shown in Appendix A, Figures A.11, A.14, A.17 and A.20.

Consideration was given to a number of factors that affect the selection and design of the concrete pipe to be used for the conduit stretching from the reservoir to the bypass channel. The four integrated facilities are located near seismically active areas and their existing subsurface conditions are similar, consisting of soft clays and peat soils overlying denser and stiffer inter-bedded sands and clays. These soft soils will be removed and replaced with suitable embankment fill material excavated from the borrow areas. The concrete pipe should be strong enough to support the induced water pressure, the vertical earth load, and any surface surcharge loads. The concrete pipe joints should be flexible enough to withstand settlements and the joints should also be water tight. The water tightness of the joints cannot be tested until the trenches are completely backfilled and the system is fully operational. Considering all these factors, a Double-Gasketed Spigot type precast reinforced concrete pipe is recommended for all the sites. The open areas on the outside and inside of pipes joints shall be filled with cement mortar during construction.

Concrete Pipe Strength Requirement

The external load acting on the pipe depends upon the type of live load, depth of cover (fill depth), and soil type. The required strength of the concrete pipe is expressed in terms of the 0.01-inch crack D-load strength and is expressed in pounds per linear foot per foot of inside diameter. The equation for D-load strength is given in Appendix A.6.

The depth of soil over each conduit pipe is given in Table 5.2 and ranges from 12.2 feet to 17 feet, depending on pipe diameter and facility location. The AASHTO HS20 vehicle load is assumed to be the major live load over the pipes. The American Concrete Pipe Association manual suggests that the effects of HS20 vehicle live loads are negligible for depths of cover greater than 9 feet. The only load that needs to be considered is the dead load of the overlying soil.

The dead load acting over each pipe was calculated assuming the overlying soil has a unit weight of 110 pounds per cubic foot. The dead loads vary depending on fill depth, which varies at each facility location, and pipe diameter. The dead loads are summarized in Table 5.2.

The load that a concrete pipe can support depends on the width of the bedding, the contact area, and the quality of the contact between the pipe and the bedding. Since the underlying soil is prone to settlement, an important consideration in selecting a bedding material is to ensure that positive contact is maintained between the bed and the pipe as settlement occurs. It is proposed that Class B bedding, a well-graded crushed rock, be used as bedding material. As the underlying soil settles, the granular materials will shift to attain positive contact with the pipe surface. The dead load factor is 1.9 and the live load factor is 1.5 for Class B bedding. The required D-load strength ($D_{0.01}$) values for the 6-foot and 8-foot diameter pipes are given in Table 5.2. The D-load strength of each pipe was multiplied by the respective factor of safety (F.S.) to get the ultimate strength (D_{ult}) of the pipes. The F.S. values used to determine the ultimate strength were adjusted based upon the D-load using the relationship specified in ASTM C655M. The 6-foot and 8-foot diameter pipes should be designed to withstand the D-load and ultimate strength values as given in Table 5.2.

Table 5.2: Summary of Concrete Conduit Pipe Design

Facility	Webb Tract, SJR River		Webb Tract, False River		Bacon Island, Middle River		Bacon Island, Santa Fe Cut	
Pipe Diameter	6 ft	8 ft	6 ft	8 ft	6 ft	8 ft	6 ft	8 ft
Embankment Elevation	+11	+11	+11	+11	+10.2	+10.2	+10.4	+10.4
Pipe Invert Elevation	-12	-12	-12	-12	-10	-10	-10	-10
Pipe Cover Depth (ft)	17	15	17	15	14.2	12.2	14.4	12.4
Dead Load (lbs)	1456	1343	1456	1343	1153	1270	1166	1283
Live Load (lbs)	0	0	0	0	0	0	0	0
D-load strength (lbs/ft)	128	88	128	88	101	84	102	84
Factor of Safety (F.S.)	1.36	1.50	1.36	1.50	1.49	1.50	1.49	1.50
Ultimate Strength (D_{ult}) (lbs/ft)	174	133	174	133	151	125	152	127
Minimum Wall Thickness (in)	6	8	6	8	6	8	6	8

5.3.4 Trash Racks and Stop Logs

To stop the flow of debris and floating particles, trash racks will be provided at both ends of the conduit pipes. The trash racks will be made of anti-fouling steel and the clear spacing between the bars should not be more than 2 inches. This spacing requirement will also prevent adult/predator-sized fish from exiting the reservoir. For easy placement and removal, the trash racks will be placed in slots. The racks will remain in place by gravity action and hence no additional anchorage is needed.

The stop log slots will be provided at each end of the conduit pipes. These slots can be used to place stop logs to stop the flow of water, so inspection and maintenance of the conduits can be carried out. Each stop log will be made out of aluminum panels.

5.3.5 Energy Dissipaters

The exit velocities at both ends of the conduit pipes are high (up to 12 ft/sec) and due to the nature of the peat soils underwater erosion may occur downstream of the outlet structures. The conduit pipe outlet structures described below were designed to dissipate energy under both submerged and un-submerged outlet conditions. This design, however, is being checked and may be modified.

Baffled apron drop structures will be used to dissipate excess energy at both ends of the conduit pipes. Drops in grade occur from the conduit outlet to the reservoir and from the conduit outlet to the bypass channel. Baffled apron drops were selected for two reasons: (1) they do not require a downstream water surface for satisfactory performance and (2) they can function under a wide variation of downstream water surface elevations. All three conduit pipes will discharge into a common energy dissipation structure. Riprap will be placed after the end of each apron. The design of the baffled apron is based on USBR specifications and is summarized in Appendix A.7. Detailed layout drawings of the energy dissipation structure for the pipe conduits are given in Appendix C, Figures C.29 through C.33.

5.4 Mechanical Engineering Design

5.4.1 General

In order to reduce plant construction costs, no oil room or maintenance bay will be provided. It is assumed that any oil purifying or major maintenance will be done at another plant. This is similar to how the South Bay Pumping Plant operates. The following sections provide details regarding the selection of mechanical equipment applicable to the pumping plant. Quantitative data on the various mechanical components are provided in Appendix A.7.

5.4.2 Pump Selection

A hydraulic analysis was performed to calculate the total dynamic head (or maximum pumping head) that the pumps must be able to operate against. The total dynamic head includes static head, pipe friction head losses, and minor head losses from valves and fittings. A summary of total dynamic head for each pumping plant is given in Table 5.3. The design methodology used, total dynamic head calculations, and head loss coefficients assumed in the analysis are provided in Appendix A.8. A variety of configurations were analyzed, based on the proposed year round operation of the facilities, to optimize the number and size of pumps and conduits needed to pump and convey a design flow of 1500 cfs. As mentioned in Section 5.3, the configuration chosen includes three 400 cfs 1500-hp pumps and two 150 cfs 800-hp pumps discharging into two 8-foot conduit pipes and one 6-foot conduit pipe, respectively.

Table 5.3: Total Dynamic Head for Each Integrated Facility Pumping Plant

		Total Dynamic Head			
Case	Flow	Webb Tract, SJR River	Webb Tract, False River	Bacon Island, Middle River	Bacon Island, Santa Fe Cut
Diversions	150 cfs	16.2	16	16.2	16.2
	400 cfs	9.9	9.7	9.9	9.9
Releases	150 cfs	35.3	34.9	35.3	35.3
	400 cfs	23	22.5	23	23

All pumping units will be vertical-type mixed flow pumps driven by a floor-mounted fixed speed motor connected to a right angle gear. The pumps were chosen to have a vertical configuration to match the formed suction intakes. The required heads dictated the choice of mixed flow pumps. (With this type of pump the power curve is such that they cannot be run with the discharge valve closed because the required power would exceed the capacity of the motor.) Floor-mounted fixed speed motors and right angle gears were chosen to drive the pumps to minimize the vertical height of the plant.

An FSI will be mounted to each pump below the impeller to eliminate vortex formation in front of the pump. FSIs require less submergence than other configurations. Other configurations would likely require the midbay floor level to be lowered to meet submergence requirements, and lowering of the midbay was minimized to reduce costs.

When the reservoir level is low and submergence requirements for the 400 cfs pumps are not met, pumping will be limited to the 150 cfs pumps. The required head for the 150 cfs pumps, then, is greater than for that of the 400 cfs pumps. Given the wide range of operating heads for the 150 cfs pumps, it will probably be necessary to choose pumps such that at maximum head the best efficiency point is to the right of the operating point. Otherwise, at minimum head, the pumps would likely experience vibration problems. Furthermore, at minimum head, both 150 cfs pumps may have to be operated in order to maintain sufficient friction head in the common 6-foot conduit pipe. Otherwise, the head would be too low and the pumps would experience vibration or cavitation problems. Under low head conditions, it may be necessary to throttle with the butterfly valves to prevent vibration or cavitation problems.

Given the planned configuration of the pumping plant, a partial vacuum may form within the piping downstream of the pumps when the pumps are shut down. An analysis should be performed to determine the maximum vacuum that may occur and the pipe thickness should be sufficient to avoid collapse. This analysis will require dynamic modeling and is beyond the scope of this feasibility study.

5.4.3 Valve Selection

Pump Discharge Valves

DWR often uses ball valves for the pump discharge valves in high pressure pumping plants but, since the pressure will be low in this pumping plant design, AWWA Class 75B butterfly valves were chosen instead. The use of check valves for the pump discharge valves were considered, but check valves would allow forward flow through the pumps and into the reservoir. This is not desired because the maximum reservoir level is lower than the maximum river level, and uncontrolled flow into the reservoir during high river stage is not acceptable. Hydraulic actuators were chosen to operate the butterfly valves, with a single hydraulic power unit provided for all of the valves in the plant. Backup hydraulic power will be provided by the HPU located in one of the conduit pipe valve vaults by running hydraulic and control lines between the vaults and the plant. On loss of power, these valves should close. It may be desirable to use either spring-to-close actuators or accumulators to provide the stored energy to close the valves; alternatively, the emergency generator would provide the power to close the valves. On loss of power, valve closure will prevent possible back flow through the pumps.

Conduit Valves

As is the case for the pump discharge valves, the conduit pipes will also have low pressure, so AWWA class 75B butterfly valves were also chosen for the conduit pipes. Hydraulic actuators were chosen to operate the butterfly valves, with a single hydraulic power unit provided for all of the valves in each vault. Backup hydraulic power will be provided by the HPU located in the pumping plant by running hydraulic and control lines between the vaults and the plant. On loss of power, these valves should close. It may be desirable to use either spring-to-close actuators or accumulators to provide the stored energy to close the valves; alternatively, the emergency generator would provide the power to close the valves. On loss of power, valve closure will prevent possible back flow through the pumps. Valve closure would also prevent possible flow from the river to the reservoir through the conduit pipes.

5.4.4 Gantry Crane

A gantry crane is required to lift the pumps, motors, and right-angle gears for maintenance. Since removal of the valves and actuators is anticipated to be required much less frequently, it was assumed that these would be moved using a mobile crane. By limiting the required access of the gantry crane to the above mentioned components, the cost of the gantry crane will be reduced.

5.4.5 Heating, Ventilation, and Air Conditioning

It is assumed that heating and air conditioning will be limited to the office, control room, and motor control room. Only ventilation will be provided to the rest of the plant. A summary of the HVAC equipment to be used in the pumping plant is provided in Table 5.4.

Table 5.4: Pumping Plant HVAC Equipment Summary

Location	Equipment	Quantity
Motor Control Room	10 Ton Cooling Only Air Conditioning Unit	2
	3.3 Kw Electric Unit Heater	1
Office	Split System Heat Pump, 1.2 Tons Cooling and 9 MBh heating	1
Control Room	Split System Heat Pump, 2.0 Tons Cooling and 1.8 MBh heating	1
Mid & Lower Levels	Ventilation Fans (5400 CFM each)	2

5.4.6 Miscellaneous

An 850 gpm 15-hp sump pump will be provided to empty the midbay area for maintenance purposes, such as dredging and cleaning the midbay, performing maintenance at the gate structures, and performing maintenance on the formed suction intakes. This cost for this sump pump will be included, but it is not shown on the feasibility level design drawings.

Combination air valves will be provided just downstream of the pump discharge valve. Each 150 cfs pump discharge pipe (4-foot-6 inch pipe) will contain a 12-inch air valve and each 400 cfs pump discharge pipe (7-foot pipe) will contain an 18-inch air valve.

5.5 Electrical Engineering Design

5.5.1 General

Feasibility level electrical engineering design for the electrical components of the integrated facility was completed and the major equipment recommendations are discussed here.

5.5.2 Transformer Sizing

A transformer sizing simulation was completed using the EDSA Micro Corporation Advanced Power Flow Program. The simulation considered only motor loads. The transformer loading consisted of the three 1500 hp motors and the two 800 hp motors at the 4160 voltage level and the results of the transformer sizing simulation indicate the need for a 7.5 MVA transformer. The 7.5 MVA transformer is at 83% of its capacity, which leaves adequate power for other low voltage loads. It is recommended that a 7.5/8.85 MVA, 230kV-4.16kV, OA/FA rated transformer be used, which will provide additional capacity with the implementation of forced air-cooling. This additional capacity could be used in any future expansion of the facility.

5.5.3 Utility Source

A PG&E area assessment was not performed in this study. The nearest utility source that can handle the In-Delta Storage project's anticipated load of 7.5MVA is estimated to be, at most, six miles from the project islands.

5.5.4 Equipment Layout

The major electrical equipment includes a control room and a switchyard containing a transformer, circuit breaker and a disconnect switch. The control room is located on the middle level of the pumping plant and has a minimum ceiling height requirement of twelve feet to accommodate the switchgear, ductwork, overhead raceway, and all other associated electrical equipment that will be installed. The switchyard is located on the embankment in front of the pumping plant valve vaults. A summary of the major electrical equipment is provided in Table 5.5.

Drawings of the pumping plant single line diagram, switchyard, control room and medium voltage switchgear enclosure arrangement are provided in Appendix C, Figures C.22 through C.26 and are typical of what will be required.

Table 5.5: Major Electrical Equipment

5kV Metal Clad Switchgear
Vacuum or SF6 Circuit Breakers
7.5/8.85 MVA, 230kV-4.16kV, OA/FA rated service transformer
Programmable Logic Controllers
Microprocessor based multifunction relay protection for the motors, switchyard equipment, and feeders
Modbus Plus communication protocol
Low Voltage Motor Control Center
Low Voltage Distribution Center

5.5.5 Recommendations

Fixed speed motors were chosen to drive the pumps. This type of motor uses across-the-line starting, which causes a large in-rush current, typically five to six times the full load amperage of the motor. This will cause stress on the motors and the feeders, which could be eliminated by the use of variable frequency drives. Therefore, it is recommended that variable frequency drives be considered. Variable frequency drives provide many advantages including energy savings, reduced equipment wear and stress, and increased efficiency. The energy savings will come from an increase in the power factor from a typical 0.85 to a minimum of 0.95 for medium voltage variable frequency drives. Modern clean power variable frequency drives reduce harmonics, are reliable and provide excellent field performance. Variable frequency drives inherently provide motor and feeder protection. Also, since the pumping head may vary greatly, more precise motor speed control may be required to operate the pumps in their optimum range. Lastly, options such as remote operation and monitoring over a network using a protocol such as Modbus plus are easily configurable with modern variable frequency drives.

An area assessment should be performed by PG&E to develop accurate distances to the nearest utility source that can handle the In-Delta Storage project's anticipated load of 7.5MVA.

Chapter 6: Bypass Channel

6.1 Introduction

The bypass channel is used to convey reservoir releases into the river and is shown in Figure 1.4. Reservoir releases enter the bypass channel at its upstream end through the conduit pipes and/or through Gate #3. The bypass channel is isolated from the fish screen facility and transition pool by a structural sheet pile wall.

6.2 Design Criteria

The following criteria were used in the design of the bypass channels.

a) The bypass channel should be designed to accommodate a maximum flow rate of 2250 cfs. Because the project site is located in the areas of tidal influences, the bypass channel should be able to pass the maximum flow during the lowest tide levels.

b) To prevent bank erosion, channel degradation, and scouring along the sheet pile wall, flow velocities within the channel should not exceed 3 feet per second (ft/sec). If the channel velocities exceed 3 ft/sec, adequate bed and slope protection should be provided.

c) Adequate freeboard should be provided within the channel to provide maximum protection during times when the maximum release flow of 2250 cfs coincides with the highest tide levels (300-year flood level).

d) Trash and other floating debris should not be allowed to enter or reside in the bypass channel.

6.3 Channel Design

The channel design included selecting channel bed elevations based on island topography and selecting the most efficient channel geometry given the overall layout of the integrated facility. The following sections outline the design procedures in more detail.

6.3.1 Bed Level

The bed level of the bypass channel on the upstream side equals the invert elevation of the outlet structures (Gate #3 and the pipe conduit). The invert elevations of the outlet structures were set according to the topography at each site. This was done to minimize the height of the structures. The adopted bypass channel bed levels for each integrated facility are given in Table 6.1.

6.3.2 Channel Geometry

To maintain embankment stability, the maximum velocity in the channel should not exceed 3 ft/sec. At the upstream end, the channel section is trapezoidal with a side slope of 3H:1V. On the downstream end, one side of the bypass channel will continue as a sloped section while the other side will consist of the vertical sheet pile wall.

Sheet Pile Wall

The bypass channel is isolated from the fish screen and transition pool area by a vertical sheet pile wall. The top of the sheet pile wall will extend to the top of the embankment. The sheet pile wall will be designed so water will not flow freely between the bypass channel and the transition pool; however, the wall will not be completely sealed. Any openings in the sheet pile wall should not be greater than 1.75 mm, which is equivalent to the maximum opening of the fish screens. This will also help to prevent juvenile fish from entering the reservoir. The top elevations of the sheet pile wall for each integrated facility are given in Table 6.1

Channel Bottom Width

The channel was designed to accommodate a maximum flow of 2250 cfs. The size of the riprap and Manning's roughness coefficient are interdependent. Manning's roughness coefficients of 0.02 and 0.025 were used for the channel bed and channel sides, respectively. The required bottom width of the channel was determined for a design discharge of 2250 cfs with both minimum and maximum slough levels. Since the bottom elevation of the channel was determined based upon the existing topography, the controlling situation occurred when the slough levels are lowest and the bypass channel is discharging the maximum flow. Initial estimates of the required channel geometry were made using the energy balance approach. Adequacies of the channel geometry were verified using the U.S Army Corps of Engineers HEC-RAS program. The bypass channel bottom width for each integrated facility is given in Table 6.1.

6.3.3 Slope Protection

Both sides, as well as the bottom of the bypass channel will be lined with rock riprap to prevent bank erosion.

6.3.4 Access Bridge and Trash Rack

The vehicle access bridge will connect the end of fish screen facility with the levee top. This bridge will allow access to the fish screen from both ends as well as allow traffic to move from one side of the facility to the other. The bridge has a deck width of 15 feet and spans across the bypass channel. The bridge is designed as a simple box culvert and has vertical abutments and intermediate piers. The piers provide support for the deck and they will be used to hold the trash racks in position. The trash racks will be made out of steel sections with clear openings of not more than 2 inches on either side of the rack. This spacing was recommended by fishery agencies to prevent the attraction and egress of adult-sized fish (specifically salmon and steelhead) into and out of the bypass channel. The trash racks will prevent the flow of debris and adult-sized fish from entering into the intake facility.

The bypass channel velocity profiles for each integrated facility are shown in Appendix A.9, Figures A.27 through A.29. The cross sectional area of the channel at the bridge location is smaller than the rest of the channel. This causes the flow velocity through the bridge section to be higher than the permissible limit (3 ft/sec) for the unlined channel. Although the box culvert bridge section is designed to withstand velocities higher than 3 ft/sec, the bypass channel sections immediately upstream and downstream of the bridge may experience scour problems. To mitigate for the higher velocities and to prevent scouring, the channel bed and channel slopes upstream and downstream of the bridge will be lined with riprap.

Table 6.1: Summary of Bypass Channel Design

<table>
<tr><th colspan="3" rowspan="2">Bypass Channel Component</th><th colspan="4">Integrated Facility Location</th></tr>
<tr><th>Webb Tract, San Joaquin River</th><th>Webb Tract, False River</th><th>Bacon Island, Middle River</th><th>Bacon Island, Santa Fe Cut</th></tr>
<tr><td colspan="3">Upstream Bed Level (ft)</td><td>-15</td><td>-16</td><td>-12</td><td>-8</td></tr>
<tr><td colspan="3">Downstream Bed Level (ft)</td><td>-16</td><td>-17</td><td>-13</td><td>-9</td></tr>
<tr><td colspan="3">Sheet Pile Wall Top Elevation (ft)</td><td>11</td><td>11</td><td>10.2</td><td>10.4</td></tr>
<tr><td colspan="3">Bottom Width (ft)</td><td>30</td><td>30</td><td>40</td><td>70</td></tr>
<tr><td rowspan="4">Side Slopes</td><td rowspan="2">Upstream End</td><td>Left Bank</td><td>3:1</td><td>3:1</td><td>3:1</td><td>3:1</td></tr>
<tr><td>Right Bank</td><td>3:1</td><td>3:1</td><td>3:1</td><td>3:1</td></tr>
<tr><td rowspan="2">Down-stream End</td><td>Left Bank</td><td>Vertical</td><td>Vertical</td><td>Vertical</td><td>Vertical</td></tr>
<tr><td>Right Bank</td><td>3:1</td><td>3:1</td><td>3:1</td><td>3:1</td></tr>
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